# Studies of Stone-Setting Mortars

United States Department of Commerce

National Bureau of Standards

Building Materials and Structures Report 139

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Daniel W. Kessler and Ross E. Anderson
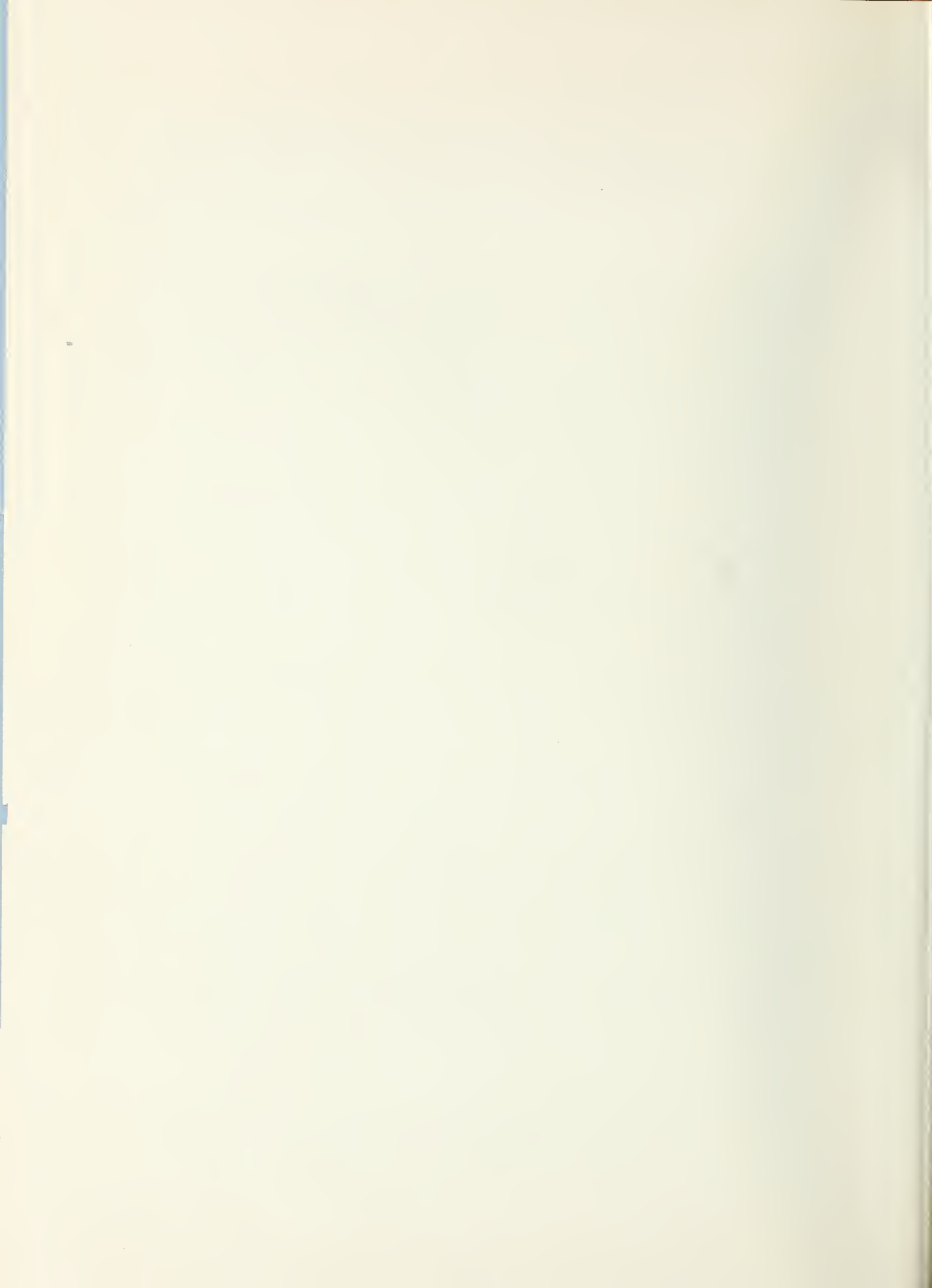
# *Foreword*

Stone masonry structures sometimes exhibit defects such as spalling at mortar joints, separation of mortar from stone in vertical joints, discoloration, and efflorescence. These are often attributed to the mortar. To determine the extent to which mortars are responsible for undesired performance in stone masonry and to serve as a guide in the selection of suitable mortars, laboratory tests on several types applied to the more common varieties of building stone were conducted. This report covers the results of these tests and includes a discussion of the relation of various properties of mortar to its performance.

A. V. Astin, *Director.*

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Several mortars were studied for bonding strengths to four types of stone, dimensional changes under three conditions of storage, and frost resistance. Higher bonding strength was obtained for mortars to fine-grained stones than to coarse-grained stones. Indications were found that mortars shrink more in a vertical direction than horizontally. If shrinkage cracks form in the specimen, the measurements will be too low. Tests on mortars cast and cured in contact with porous materials gave higher strengths and lower absorptions than were obtained on specimens cured in nonabsorptive molds. Ground limestone was found to be a satisfactory plasticizing agent. Some studies were made of mortars in structures and certain recommendations are made for selecting mortars for use in stone masonry.

## 1. Introduction

When the stone-exposure test wall [1] was being planned, considerable thought was given to the selection of mortars. One objective was to study discolorations on stone with respect to type of setting mortar. Since lime mortars and portland-cement mortars were under consideration, it was decided to set one-half of the wall in straight lime mortar and the other half in portland-cement mortar with ground limestone as a plasticizer. Little information was available on the performance of mortars containing ground limestone, and it was decided to make a study of such mixtures in comparison with mixtures without limestone.

Observations on the performance of materials in structures have indicated some of the troubles in stone masonry are: (1) Bond failure, especially in vertical joints; (2) spalling of the stone at the joints; (3) discolorations of the stone; (4) crack formations due to calcareous growths starting where lime solutions evaporate through small openings.

As pointed out by Pearson [1],[2] there are many properties of mortars which are important, but the writers considered that bonding strength, shrinkage, and durability were of most concern. There is little published information on bonding strength of mortars to stone, and most of the studies on shrinkage have been confined to linear changes that occur after the mortars have set. The curing conditions in masonry are considerably different from those commonly used in the laboratory so a few comparisons of test results for both conditions of curing were made.

Such considerations led to a series of experiments to develop data on the following points: (1) The bonding strength of various mortars to the more common types of stone; (2) comparison of the bonding strength of mortars in specimens made to simulate the conditions in horizontal and vertical joints; (3) the relative merits of two plasticizing agents, hydrated lime and ground limestone; (4) the physical properties of mortars cured under conditions similar to those in masonry as compared to those cured under standard laboratory conditions; (5) the total shrinkage of mortars in contact with porous materials; (6) the extent of volume changes in mortars under various conditions; (7) the frost resistance of various mortar mixtures.

Published data on shrinkage of cement products show a wide range in results obtained by different investigators for similar mixtures and results obtained in this study were considerably higher than many values previously reported. Carlson [2] has developed a theory of the mechanics of early shrinkage, which may explain some of the variable results. According to his theory, shrinkage cracks form in the cement paste surrounding aggregate particles because the compressibility of the particles is less than the shrinkage of the paste. Such shrinkage is not determined by the usual measurements because it does not change the overall dimensions of specimens. Some experiments were made on clay specimens to demonstrate the effect of pronounced cracking on shrinkage measurements.

Shrinkage measurements were made under conditions that were intended to give some indication of the total contractions that may occur in masonry joints and hence the results are considerably higher than many reported for the usual laboratory storage conditions. It is doubtful if these results are as high as actual shrinkages that occur in vertical joints, filled by pouring in a grout, as is frequently done in setting large ashlar blocks.

## 2. Materials

### 2.1. Mortar Ingredients

Seven brands of portland cement, including 5 gray and 2 white cements, 2 masonry cements, 1 natural cement, and a high-calcium hydrated lime were used in various mortars. Ground limestone

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[1] National Bureau of Standards Building Materials and Structures Report BMS 125.
[2] Figures in brackets indicate the literature references at the end of this paper.

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passing a No. 100 sieve was used in a few mixtures as a plasticizer. In most mixes, Potomac River sand passing a No. 8 sieve was used. Percentages retained on the Nos. 16, 30, 50, and 100 sieves were 12, 20, 49, and 95, respectively. In one group of bond test specimens, crushed marble graded between sieves Nos. 20 and 100 was used, and the silica sand, described in section 8, was used in a few mortars for experiments on different curing conditions.

In the tables and graphs the cements are designated by letters which are included in statements of proportions of the mortars. The various cements are designated as follows:

*Portland cements.* Brands: A, portland, type I; B, white portland, regular; C, white portland, air entraining; D, portland, type I; E, portland, type II; F, portland, type II; G, portland, type I.

*Masonry cements.* Brands: H, buff masonry with water repellant; I, portland with limestone addition.

*Natural cement.* Brand: J, dark gray natural.

For hydrated lime and ground limestone, the symbols (HL) and (Ls) have been used, respectively, to designate these admixtures.

## 2.2. Mortars

The mortars used in various tests are listed in table 1.

The mortar ingredients were proportioned by volume on the basis of the following weights per cubic foot: portland cement, 94 lb; masonry cement, 70 lb; natural cement, 80 lb; hydrated lime, 40 lb; ground limestone, 80 lb; silica sand, 97 lb; and crushed marble, 92 lb. The amount of water for each mix was determined by flow-table tests (ASTM C230–49T). A flow of 120 was judged to be similar to that commonly used by stone setters and this flow was used for all mixtures. The water-cement ratios of the mortars are shown in table 1.

The specimens for compressive strength, absorption, and bulk density were 2-in. cubes made and cured 28 days in accordance with ASTM designation C91–44.

TABLE 1. *Mortar designations and test values*

| Designations | | Proportions by volume | W/C [2] by weight | Test values [3] | | |
|---|---|---|---|---|---|---|
| Mortar No. | Cement brand [1] | | | Compressive strength | Absorption 48 hr by weight | Bulk density |
| 1:3 cement-sand mixtures | | | | | | |
| | | | | *psi* | *%* | *g/cm[3]* |
| 1 | A | 1(A):3 | 0.65 | ---------- | ---------- | ---------- |
| 2 | B | 1(B):3 | .66 | 4,080 | 9.7 | 1.96 |
| 3 | C | 1(C):3 | .63 | | | |
| 4 | D | 1(D):3 | .59 | ---------- | ---------- | ---------- |
| 5 | E | 1(E):3 | .66 | 2,960 | 7.6 | 2.02 |
| 6 | F | 1(F):3 | .66 | 3,370 | 9.3 | 1.98 |
| 7 | H | 1(H):3 | .86 | 380 | 12.7 | 1.66 |
| 8 | I | 1(I):3 | .63 | 1,150 | 11.7 | 1.77 |
| 9 | J | 1(J):3 | .78 | 120 | 13.6 | 1.68 |
| Cement, lime, and sand mixtures | | | | | | |
| 10 | A | 1(A):0.1(HL):3 | 0.60 | ---------- | ---------- | ---------- |
| 11 | B | 1(B):0.1(HL):3 | .69 | 4,260 | 9.6 | 1.96 |
| 12 | C | 1(C):0.1(HL):3 | .67 | 3,500 | 8.7 | 1.91 |
| 13 | F | 1(F):0.1(HL):3 | .67 | 3,530 | 9.2 | 1.98 |
| 14 | G | 1(G):0.1(HL):3 | .67 | 3,690 | 9.8 | 1.89 |
| 15 | E | 1(E):0.4(HL):3 | .71 | 4,760 | 9.3 | 1.99 |
| 16 | F | 1(F):0.4(HL):3 | .71 | 3,840 | 9.6 | 1.97 |
| 17 | F | 1(F):0.5(HL):3 | .74 | 3,820 | 8.5 | 1.96 |
| 18 | F | 1(F):1(HL):3 | .79 | ---------- | ---------- | ---------- |
| Cement, ground–limestone, and sand mixtures | | | | | | |
| 19 | F | 1(F):0.5(Ls):3 | 0.70 | 4,420 | 8.8 | 2.00 |
| 20 | F | 1(F):0.6(Ls):2.4 | .60 | ---------- | ---------- | ---------- |
| 21 | F | 1(F):1.2(Ls):2.4 | .70 | ---------- | ---------- | ---------- |
| 22 | F | 1(F):2(Ls):2 | .69 | 4,380 | 9.1 | 1.98 |
| Portland and masonry or natural-cement mixture | | | | | | |
| 23 | D & H | 1(D):1(H):6 | 0.65 | ---------- | ---------- | ---------- |
| 24 | E & H | 1(E):1(H):6 | .70 | ---------- | ---------- | ---------- |
| 25 | F & H | 1(F):1(H):6 | .73 | 1,600 | 11.3 | 1.76 |
| 26 | F & J | 1(F):1(J):6 | .69 | 1,460 | 10.6 | 1.81 |
| 27 | G & J | 1(G):1(J):6 | .65 | ---------- | ---------- | ---------- |

[1] The cements are described in section 2.2.
[2] The ratios of weights of water to cement, termed W/C, were based on the weights of portland, masonry, or natural cement used in the various mixtures. When 2 cements were used in 1 mortar the value of C was the sum of the 2 weights.

[3] Two-inch cubes were made and tested for compression in accordance with ASTM Designation C91–44T. Absorption and bulk density were determined in accordance with ASTM Designation C97–47.

## 2.3. Stones Used in Bond Tests

In making bond test specimens, the four types of natural stone listed in table 2 were used in slab form as follows: Limestone from Rockwood, Ala.; sandstone from Amherst, Ohio; marble from Danby, Vt.; granite from three sources.

The limestone was of the oolitic variety, of medium texture, and contained about 98 percent of calcium carbonate. The sandstone was of medium texture, rather high porosity, and contained approximately 97 percent of silica. The marble was of medium texture and was typical of the white building grades. Its composition was about 99 percent of calcite. The granites varied from medium to coarse and all were of the biotite variety. It should be noted that textures in the different types of stone are relative to each other only within a particular type. For instance, medium-textured granites are much coarser grained than medium-textured marbles, sandstones, and limestones. The range in grain sizes for the stones used was approximately as follows: Limestone, 0.25 to 1 mm; sandstone, 0.1 to 0.5 mm; marble, 0.2 to 0.6 mm; and granites, 1 to 10 mm.

The surface finish on the limestone, sandstone, and marble was the same, and was obtained by grinding with No. 60 abrasive grain. This gave practically the same finish as that called sand-rubbed finish in the stone trade. The surfaces on the granite slabs varied from fine pointed to coarse hammered. These surfaces were on the blocks when received from finishing plants and were believed to be more typical of bed-joint faces commonly used in granite ashlar masonry than could have been obtained by grinding. In all cases, the two granite slabs forming one bond specimen were cut from the same block and had the same kind of finish in contact with the mortar.

Physical properties of the various stones used in making bond test specimens are shown in table 2.

TABLE 2. *Physical properties of stones used in bond tests*

| Stone | Type | Compressive strength | Modulus of rupture | Tensile strength | Absorption 48 hr by weight | Initial rate of absorption [1] | Unit weight |
|---|---|---|---|---|---|---|---|
| | | *psi* | *psi* | *psi* | *%* | *g/min* | *lb/ft³* |
| 1____ | Limestone____ | 7, 400 | 1, 020 | 360 | 4. 7 | 9. 8 | 145 |
| 2____ | Sandstone____ | 8, 600 | ____ | ____ | 5. 8 | 18. 0 | 135 |
| 3____ | Marble____ | 12, 200 | 1, 550 | 700 | 0. 10 | 0. 2 | 170 |
| 4____ | Granite____ | 24, 900 | ____ | ____ | . 29 | . 3 | 164 |
| 5____ | ____do____ | 20, 500 | ____ | ____ | . 14 | . 3 | 169 |
| 6____ | ____do____ | 19, 000 | ____ | ____ | . 10 | . 3 | 167 |

[1] Grams of water absorbed in 1 min when a 30-in.² face of a dry specimen is standing in water to a depth of ⅛ in., ASTM Designation C67–44.

# 3. Bond Tests

## 3.1. Specimens

Bond test specimens were made with ¼-in. mortar joints between slabs of stone. The stone slabs were 4 by 6 in. on the face, and thicknesses varied from 1 to 2 in. Before the joints were made the stone slabs were soaked in water for 30 min, freed of surface water, and dried in air for 10 min. The joints were made in a special rack shown in figure 1. In making specimens to simulate the conditions in horizontal joints, the lower slab was first clamped as shown in the lower part of figure 1. A square metal rim 4 by 4 in. inside and ⅜ in. deep was centered over the lower stone



FIGURE 1. *Rack used in preparing bond test specimens.*

and filled level full of mortar. The rim was then removed, after which the top slab was centered above, in crossed position, and forced down until it came in contact with two metal strips on the rack, which gaged the joint thickness to ¼ in. Specimens made to simulate vertical masonry joints were made in another part of the rack shown in the upper portion of figure 1. On one slab four ¼-in. metal cubes were sealed for thickness spacers and also to prevent the slabs from being drawn together by shrinkage of the mortar. Two slabs were clamped together on the spacers in the position shown. The lower edge and two sides were closed by wooden strips coated with grease on the joint faces. As a precautionary measure, these slabs were fastened together with very strong rubber bands that were left on until the specimens were stored. The mortar in the vertical joints was tamped in three layers with a metal bar ⅛ in. thick. Specimens for comparing the strength in horizontal and vertical joints were made in groups of six, all from one batch of mortar. Specimens for comparing the strength in the two types of joints were made mostly of limestone but a few were of sandstone and marble. As soon as a group was completed (about 30 min) the specimens were unclamped from the rack and placed on shelves in a room in which the temperature varied from 68° to 72° F and relative humidity from 45 to 65 percent.

At ages of 1, 5, 11, 18, and 25 days, the specimens were immersed for 15 min in water. Strength tests were made at 28 days.

### 3.2. Apparatus and Procedure

Tests were made with a hand-operated apparatus which consisted of a loading device shown in figure 2, and a pendulum-type scale with a capacity of 1,400 lb and a sensitivity of 1 lb at all loads. Specimens were supported on knife edges of the rocker type and the supporting edges made contact with the ends of the stone slabs about ¼ in. from the edges of the joints. A special tool was made for transmitting the loads. This consisted of two metal edges 4¾ in. apart, rigidly attached to a connector as shown. Between the tool and the loading mechanism was a spherical seat for accurate adjustment. Loads were applied at a rate of approximately 100 lb/min.

### 3.3. Results and Discussion

The results of bond tests are shown in tables 3 to 6, along with results of compression tests, absorption, and bulk density determinations on mortar cubes. Figures 3, 4, 5, and 6 show various types of breaks. In these figures the letters L, S, M, and G, denote limestone, sandstone, marble, and granite, respectively. Numerals following these letters denote different specimens, the slab
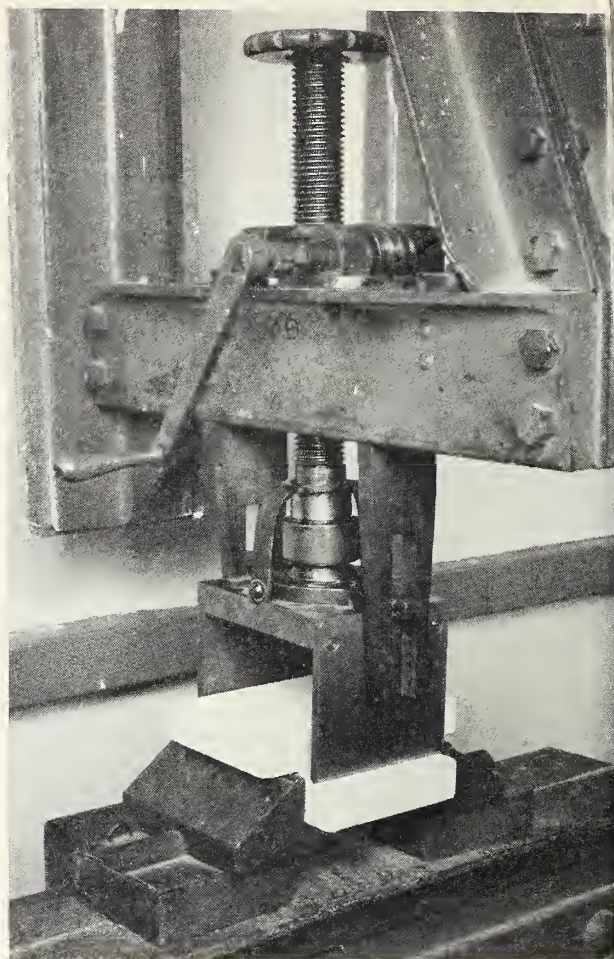


FIGURE 2. *Hand-operated machine used in determining bonding strength of mortars to stone.*

with the mark being one part and that directly above, its companion. It will be noted in some cases, that the mortar pulled out numerous particles of stone and in one sandstone specimen (S4, fig. 4) the break was entirely in the stone. In some instances the ends of the stone slabs were broken off before the bond failed. The data in tables 3 to 6 show the average bonding strengths for groups, usually of three specimens. When one or more specimens of a group failed by breakage of the stone before the bond was ruptured, the averages for such groups were obtained by using the highest bonding stress developed in the specimens that did not break the bond, along with the actual bond value of the remaining specimens. Such averages are somewhat lower than the actual bonding value. A few specimens failed by breaking partly or entirely in the mortar due to the tensile strength of the mortar being lower than the bonding strength. This type of failure is shown in figure 3 by specimen 12.
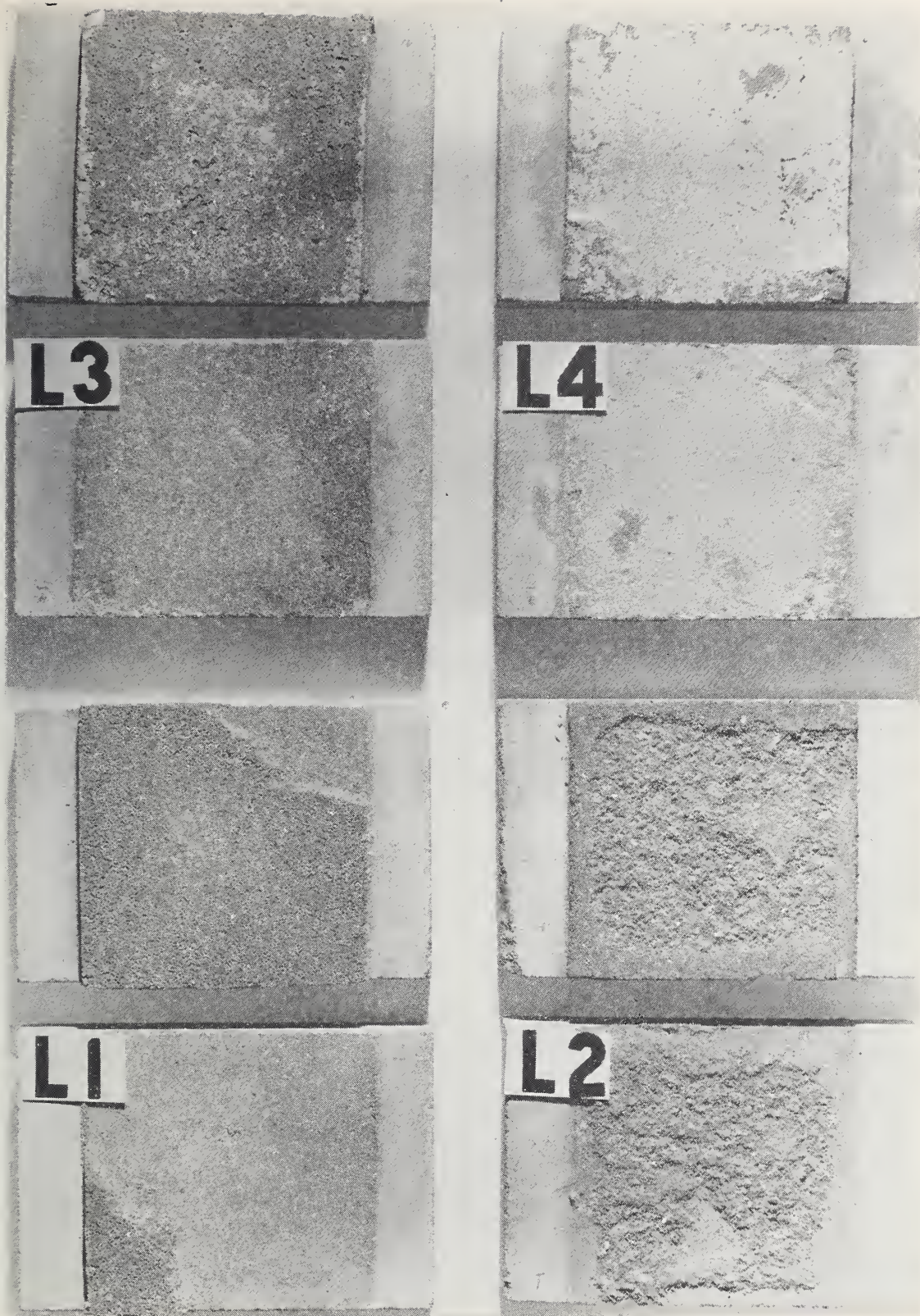
FIGURE 3. *Limestone bond specimens after test showing types of failures.*
Each numbered slab and the one immediately above formed the original test specimen.
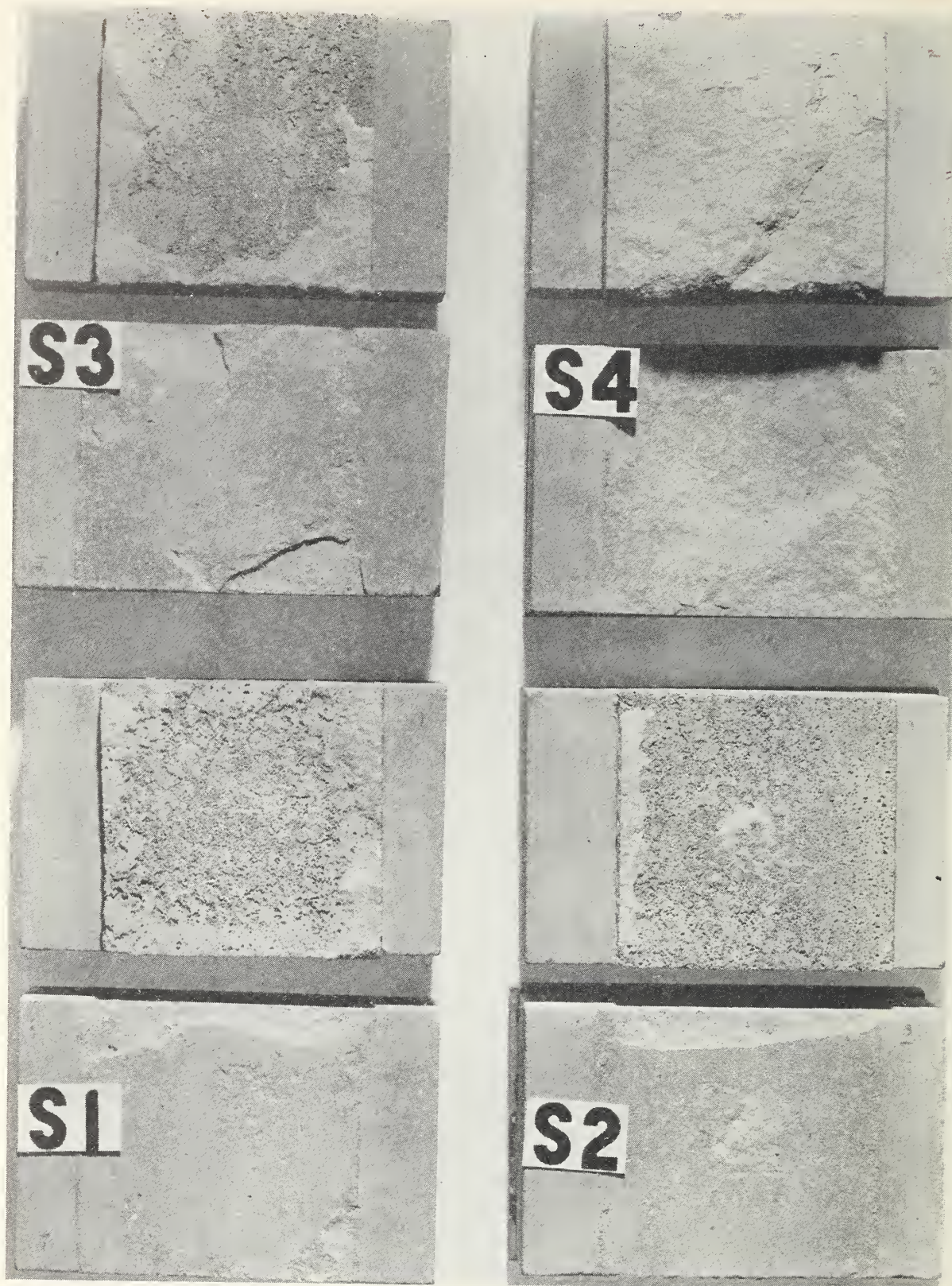
FIGURE 4. *Sandstone bond specimens after test showing types of failures.*
Each numbered slab and the one immediately above formed the original test specimen.

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FIGURE 5. *Marble bond specimens after test showing types of failures.*
Each numbered slab and the one immediately above formed the original specimen.

FIGURE 6.  *Granite bond specimens after test showing types of failures.*
Each numbered slab and the one immediately above formed the original specimen.

TABLE 3. *Results of tests on mortars for bonding strength to limestone*

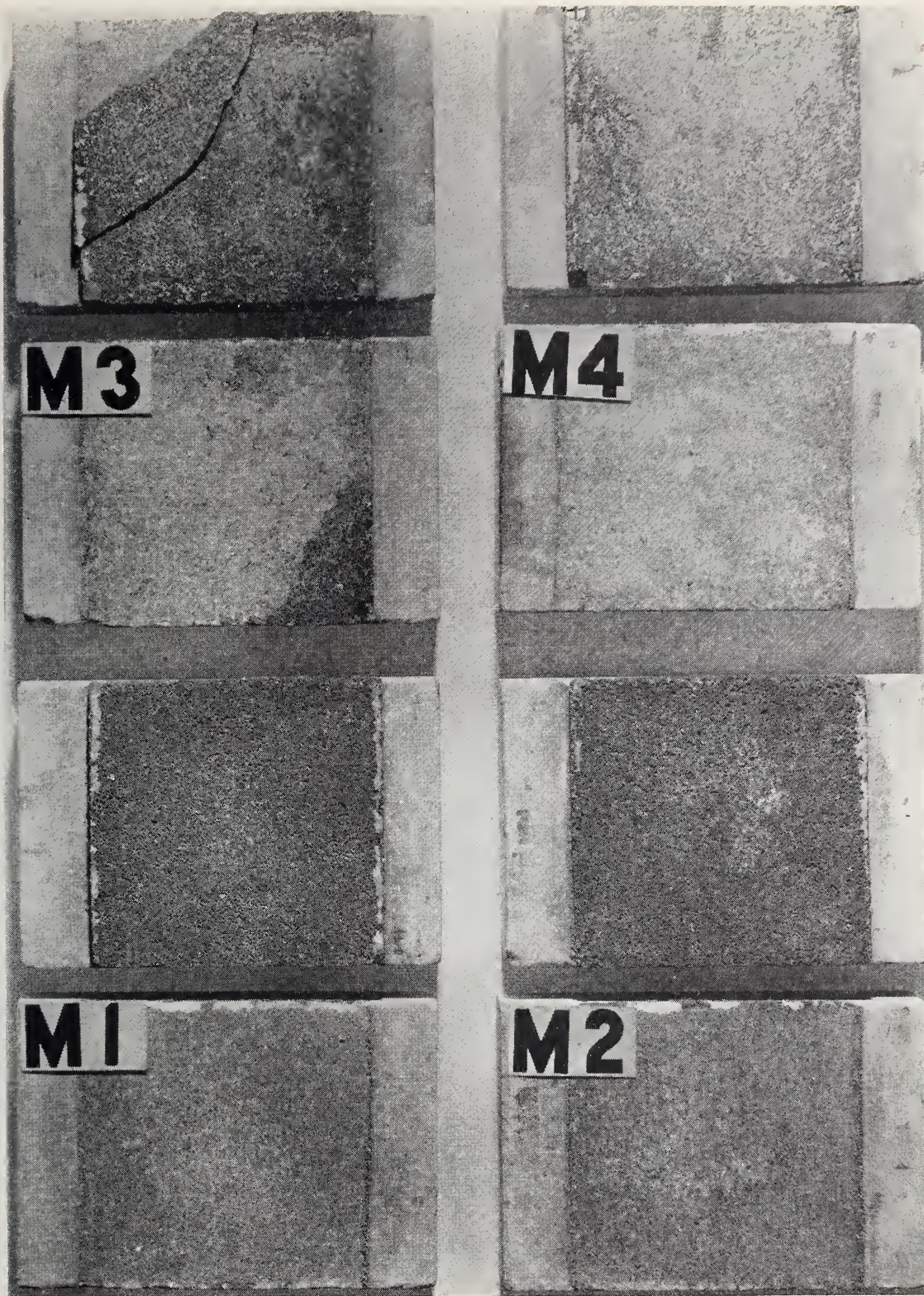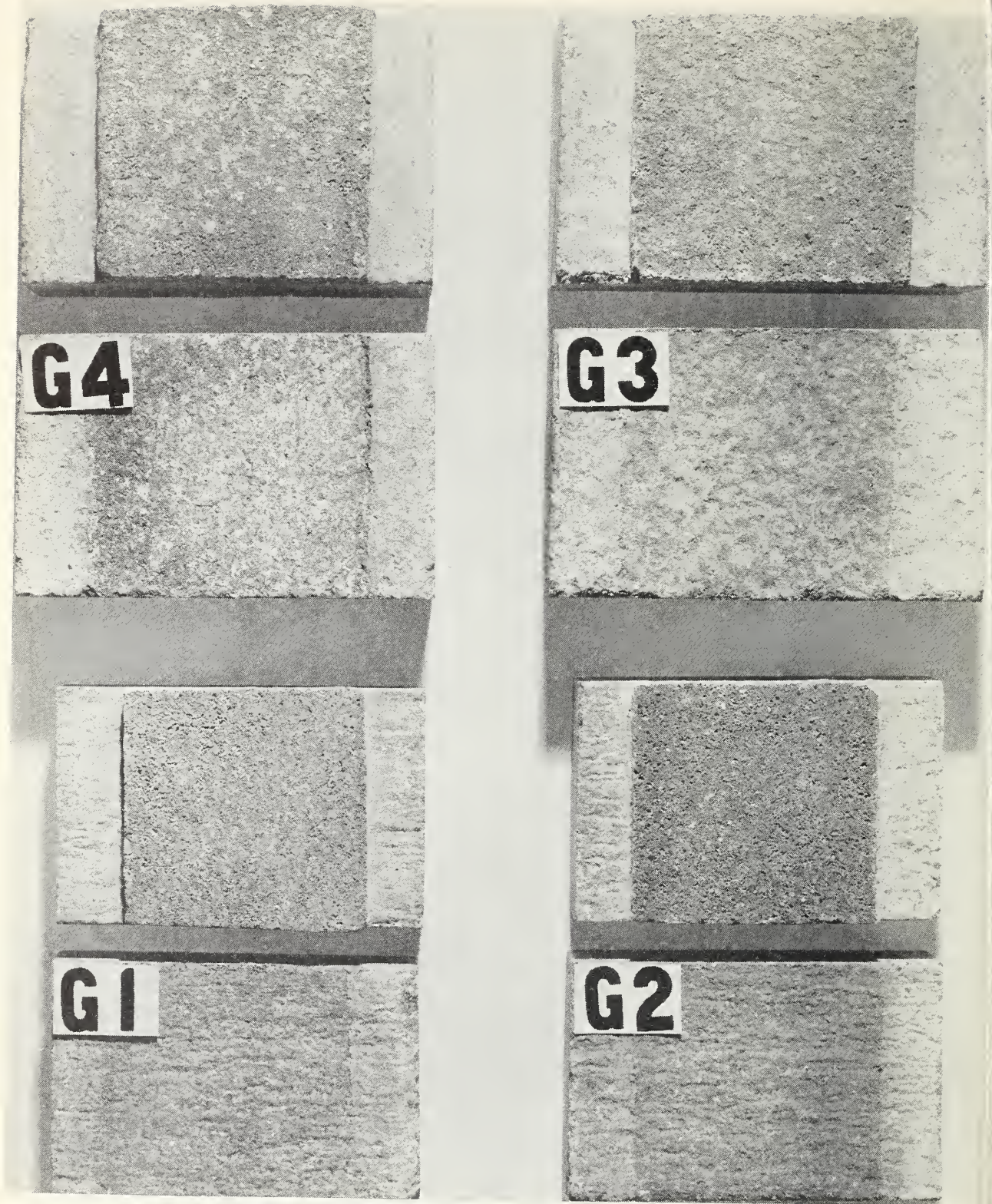| Mortar No. | Mortar mixtures | | Bond tests on limestone | | | | | Tests on mortar cubes | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Dry ingredients | W/C [1] | Type of joint [2] | Average tensile strength | Type of break for specimen [3] | | | Compressive strength | Absorption 48 hr by weight | Bulk density |
| | | | | | 1 | 2 | 3 | | | |
| | | | | *psi* | | | | *psi* | *%* | *g/cm³* |
| 6 | 1(F):3 | 0.66 | H | 60 | a2 | a8 | e | 3,700 | 9.9 | 1.97 |
| 6 | do | .66 | V | 36 | a4 | a9 | a3 | | | |
| 6 | do | .66 | H | 70 | a10 | a8 | a6 | | | |
| 2 | 1(B):3 | .66 | H | 52 | e | a12 | a8 | 4,080 | 9.7 | 1.96 |
| 2 | do | .66 | V | 36 | a8 | a5 | a5 | | | |
| 2 | do | .66 | H | 58 | a10 | a11 | a11 | | | |
| 8 | 1(I):3 | .63 | H | 33 | a2, b1 | a1, b2 | a1, b1 | 1,150 | 11.7 | 1.77 |
| 8 | do | .63 | V | 33 | a1, b1 | a1, b1 | a1, b2 | | | |
| 7 | 1(H):3 | .86 | H | 27 | --- | --- | --- | 380 | 12.7 | 1.66 |
| 7 | do | .86 | V | 27 | b2 | b1 | b1 | | | |
| 9 | 1(J):3 | .78 | H | 21 | b14 | b14 | b13 | 120 | 13.6 | 1.68 |
| 9 | do | .78 | V | 22 | b14 | b13 | b13 | | | |
| 13 | 1(F):0.1(HL):3 | .67 | H | 47 | a3 | a5 | a3 | 3,650 | 9.8 | 1.96 |
| 13 | do | .67 | V | 51 | a5 | a5 | a4 | | | |
| 16 | 1(F):0.4(HL):3 | .71 | H | 56 | a12 | a10 | e | 4,090 | 10.0 | 1.95 |
| 16 | do | .71 | V | 54 | a5 | a5, b10 | e | | | |
| 17 | 1(F):0.5(HL):3 | .74 | H | 72 | a15 | e | a14 | 3,820 | 8.5 | 1.96 |
| 17 | do | .74 | V | 70 | a15 | a15 | a15 | | | |
| 11 | 1(B):0.1(HL):3 | .69 | H | 60 | a7, b7 | a8 | e | 4,260 | 9.6 | 1.96 |
| 11 | do | .69 | V | 38 | a6 | a2 | a3 | | | |
| 11 | do | .69 | H | 50 | a12 | a8 | a10 | | | |
| 14 | 1(G):0.1(HL):3 | .67 | H | 38 | a2 | a1 | a1 | 3,690 | 9.8 | 1.89 |
| 14 | do | .67 | V | 50 | a3, b1 | e | a4 | | | |
| 12 | 1(C):0.1(HL):3 | .67 | H | 62 | a10, 2b | a8 | | 3,500 | 8.7 | 1.91 |
| 12 | do | .67 | V | 41 | a4, b1 | a4 | a6, b2 | | | |
| 12 | do | .67 | H | 67 | a4 | a5 | e | | | |
| 16 | 1(F):0.4(HL):3 | .71 | H | 69 | a14 | a15 | a8 | 3,880 | 8.9 | 2.00 |
| 16 | do | .71 | V | 66 | a12 | e | a11 | | | |
| 19 | 1(F):0.5(Ls):3 | .70 | H | 47 | a2 | e | a1 | 4,420 | 8.8 | 2.00 |
| 19 | do | .70 | V | 51 | a11 | a5 | a6 | | | |
| 19 | do | .70 | H | 62 | a3 | a2 | a1 | | | |
| 22 | 1(F):2(Ls):2 | .69 | H | 71 | e | a12 | a4 | 4,380 | 9.1 | 1.98 |
| 22 | do | .69 | V | 45 | a5 | a6 | a3 | | | |
| 25 | 1(F):1(H):6 | .73 | H | 31 | a2 | a3 | a1 | 1,600 | 11.3 | 1.76 |
| 25 | do | .73 | V | 29 | a3, b2 | a1, b2 | a4 | | | |
| 26 | 1(F):1(J):6 | .69 | H | 36 | a3, b3 | a1, b1 | a1 | 1,470 | 10.9 | 1.78 |
| 26 | do | .69 | V | 30 | a1, b2 | a4, b1 | a1, b2 | | | |

[1] Water-cement weight ratio.
[2] H signifies horizontal joint; V, vertical joint.
[3] Types of failures in the bond tests: "a" signifies that part of the stone was pulled out by the mortar as in figure 5, M² and M⁴; "b" signifies failure of the mortar in tension as in figures 3 and 12; "e" signifies complete failure of the stone as in figure 4, S⁴, or by breaking ends of slabs. Numerals following the letters "a" or "b" denote the number of square inches of failure of the type designated by the letters. When one or more specimens of a group failed without breaking the bond, the average values in the table were obtained by using the highest bonding stress developed by such specimens and are somewhat lower than the actual bond strength. When no notes are made on the type of failure, it will be understood that the breaks in that group of specimens were entirely bond failures.

TABLE 4. *Results of tests on mortars for bond to sandstone*

| Mortar No. | Mortar mixtures | | Bond tests on sandstone | | | | | Tests on mortar cubes | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Dry ingredients | W/C | Type of joint | Average tensile strength | Type of break for specimens [1] | | | Compressive strength | Absorption 48 hr by weight | Bulk density |
| | | | | | 1 | 2 | 3 | | | |
| | | | | *psi* | | | | *psi* | *%* | *g/cm³* |
| 6 | 1(F):3 | 0.66 | H | 76 | a5 | c3 | a3 | 3,300 | 8.8 | 2.00 |
| 13 | 1(F):0.1(HL):3 | .67 | H | 66 | e | e | e | 3,580 | 8.8 | 1.99 |
| 13 | do | .67 | V | 70 | c | e | c | | | |
| 16 | 1(F):0.4(HL):3 | .71 | H | 63 | a10 | c | a15 | 3,700 | 9.7 | 1.96 |
| 26 | 1(F):1(J):6 | .69 | H | 52 | a4 | a4 | a3 | 1,390 | 10.4 | 1.81 |

[1] See footnote 3 of table 1.

TABLE 5. *Results of tests on mortars for bond to marble*

| Mortar No. | Mortar mixtures | | Bond tests on marble | | | | | Tests on mortar cubes | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Dry ingredients | W/C | Type of joint | Average tensile strength | Type of break for specimens [1] | | | Compressive strength | Absorption 48 hr by weight | Bulk density |
| | | | | | 1 | 2 | 3 | | | |
| | | | | *psi* | | | | *psi* | *%* | *g/cm³* |
| 6 | 1(F):3 | 0.66 | H | 55 | e | a4 | e | 3,310 | 8.8 | 2.00 |
| 13 | 1(F):0.1(HL):3 | .67 | H | 70 | a7 | a10 | e | 3,510 | 8.9 | 2.00 |
| 13 | do | .67 | V | 66 | a8 | a10 | a10 | | | |
| 16 | 1(F):0.4(HL):3 | .71 | H | 65 | e | e | a13 | 3,880 | 8.9 | 2.00 |
| 26 | 1(F):1(J):6 | .69 | H | 40 | a1 | a1 | a1 | 1,300 | 10.8 | 1.80 |

[1] See footnote 3 of table 1.

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Table 6. Results of tests on mortars for bond to granite

| Mortar No. | Mortar mixtures | | Bond tests on granite | | | | | Tests on mortar cubes | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Dry ingredients | W/C | Type of joint | Average tensile strength | Type of break for specimens [1] | | | Compressive strength | Absorption 48 hr by weight | Bulk density |
| | | | | | 1 | 2 | 3 | | | |
| | | | | psi | | | | psi | % | g/cm³ |
| 6 | 1(F):3 | 0.66 | H | 41 | | | | 3,180 | 9.6 | 1.96 |
| 13 | 1(F):0.1(HL):3 | .67 | H | 33 | | | | 3,400 | 9.2 | 1.97 |
| 16 | 1(F):0.4(HL):3 | .71 | H | 52 | a1 | | | 3,700 | 9.7 | 1.96 |
| 26 | 1(F):1(J):6 | .69 | H | 20 | | | | 1,700 | 10.2 | 1.87 |

[1] See footnote 3 of table 1.

When stone particles over a considerable area are pulled out by the mortar, one might assume that the tensile strength of the stone was lower than the bonding value of the mortar. Tests on marble and limestone, however, show that their tensile values are several times higher than any of the stresses developed in these tests. It seems more likely that some of the stone particles were loosened in cutting, or by some unknown physical action at the interface.

The various types of breaks that occurred in the bond tests are described in tables 3 to 6 by means of symbols and footnotes. These may be used to evaluate the averages in column 4.

Although the number of tests on sandstone, marble, and granite is probably too limited to justify definite conclusions in regard to comparative bond strengths, there are enough results to indicate that the strength of bond to granite is lower than that of the other three types. This is surprising in view of the fact that the surfaces of the granite blocks were comparatively rough. The bond between mortars and sandstone or marble appears to be considerably higher than for limestone. Such results cannot be ascribed to porosity or rates of absorption of the stones. Studies by Palmer and Parsons [3] on bonding

strength of mortars to brick indicate that bricks absorbing about 20 g of water per min when laid flat (4- by 8-in. face) in ⅛ in. of water give higher bonding strengths than bricks of either higher or lower absorption rates.[3] The absorption values of the stones used in these tests determined on the same basis were as follows: Limestone, 9.8; sandstone, 18; marble, 0.2; and granite, 0.3 g. There seems to be some correlation between grain size and bonding strength in that the finest-grained materials showed high values and the coarse-grained materials, low values. This might be interpreted to mean that the highest bond is along the grain boundaries.

Because of the uncontrollable variables mentioned above, it is impossible to reach definite conclusions from this limited series of tests in regard to mortars. Because there is little available information on bonding strength of mortars to stone, a few tentative statements based on the results obtained from the limestone specimens may be desirable.

Two 1:3 mixtures of portland cement and sand gave considerably less bonding strength in vertical joints than in horizontal joints, whereas three similar mixtures gave practically the same bonding strength in both types of joints. In four

[3] The data presented by Palmer and Parsons [3] have been considered as proving that brick units of very low rates of absorption (1 g or less per min) give relatively low strength of bond in tension in brick-mortar assemblages compared with brick of 22 g rate of absorption. Field observations and subsequent tests have caused these relations to be questioned. The following tabulation of heretofore unpublished data has been made available through the courtesy of J. W. McBurney: These data represent strength of bond of brick-mortar assemblages in tension (crossed brick) and in shear (3-brick assemblages) laid in 1:¼:3 cement-lime-sand mortar. In these tests the bricks were placed on the bed joints within 10 sec of spreading the (bed) joints. Each value for strength of bond in tension or in shear is the average of 14 tests made at the age of 28 days.

| Brick No. | Initial rate of absorption | Average tensile strength | Average shearing strength |
|---|---|---|---|
| | g/min | psi | psi |
| 1 | 0.5 | 78 | 305 |
| 2 | 2.0 | | 297 |
| 3 | 8.0 | 107 | 303 |
| 6 | 21 | 27 | 126 |
| 7 | 23 | 42 | 184 |
| 8 | 33 | 60 | 178 |
| 9 | 33 | 33 | 140 |

Certain of these tests in tension were repeated with the difference that the time of placing the second brick on the mortar joint was delayed for 30, 60, and 120 seconds, respectively. The results were as follows:

| Brick No. | Initial rate of absorption | Strength of bond in tension— | | | |
|---|---|---|---|---|---|
| | | 10 sec | 30 sec | 60 sec | 120 sec |
| | g/min | psi | psi | psi | psi |
| 1 | 0.5 | 78 | 89 | 78 | 89 |
| 3 | 8.0 | 107 | 71 | 70 | 78 |
| 9 | 33 | 33 | 57 | 34 | *5 |

* Two out of 7 assemblages had zero bond.

Note that brick 3 was lowered from 107 to 71 psi by the 30-sec delay in laying. Variations in brick 1 for all tests and brick 3 for all except the 10-sec time are not considered significant. Brick 9 withstood delay up to 1 min, but a 2-min delay practically destroyed the bond. The data in general confirm the conclusion that good bond strength can be obtained from units of very low rates of absorption.

mortars (Nos. 11, 12, 13, 14) containing a small addition of hydrated lime, the bonding values were variable with two showing lower strength in the vertical joints. All mixtures containing additions of 0.4 or 0.5 part of either the lime or ground limestone gave practically the same strength in both types of joints. The unusual mixture 1:2 (Ls):2 gave remarkably high bonding strength in the horizontal joints but was considerably weaker in the vertical joints. Masonry cements and the natural cement showed relatively low bonding strength, and in some specimens the mortars broke in tension. Half-and-half mixtures of masonry cements with portland cement in 1:3 mortars showed only slightly higher bonding strength than when only the masonry cements were used in such mixtures.

It can be inferred from these results that mortars appearing to have good working qualities are more reliable in vertical joints than are the straight portland-cement mortars. The mortars should not stiffen so rapidly that it cannot adjust itself to the early shrinkage horizontally.

# 4. Linear Dimensional Changes

## 4.1. Mortar Specimens

The mortars used for determining linear changes are listed in table 1 and also in figure 7.

One specimen 24 by 4 by 2.5 in. deep was made of each mixture. The batches were too large to be mixed strictly in accordance with ASTM Method C91–44 so they were divided into two equal parts, and mixed simultaneously by two men. After the specified manipulation the two parts were combined and worked for 1 min more by one man.

The mold consisted of a metal rim and a plaster of paris base. The base slab was 2 in. longer and 2 in. wider than the metal part. The metal rim had hinges on two sides so it could be removed without damaging the specimen. On one side of the rim were two notches that served as a guide for a device used in setting the reference pins in the mortar while plastic.

Before the mixture was placed in the mold, one thickness of blotting paper was dipped in water, freed of excess water by shaking, and laid on the plaster slab. The metal rim of the mold was then centered over the slab and blotter, after which the mortar was scooped in, tamped in three layers, and leveled. The same plaster slab was used for all specimens, but it had time to dry at least 2 days in the laboratory between specimens.

The reference plugs were cut into 1-in. lengths from a ¼-in. square brass bar. One end was then polished and marked with a cross; the other end was pointed. Guide brackets with square holes at one end were clamped into position and the reference plugs forced through the holes into the mortar. The tops of the plugs were left projecting slightly above the surface of the specimen.

## 4.2. Apparatus and Test Procedure

The measuring device consisted of two micrometer microscopes mounted 51 cm (20 in.) apart on a horizontal shaft that could be rotated through an angle of 45 degrees. A graduated invar bar was mounted parallel to the shaft and far enough forward to avoid interference with the line of sight when taking readings on the plugs. The microscopes were first focused on the scale of the invar bar and the shaft then revolved until the plugs came into line. The reference marks were then brought into focus with leveling screws under the specimen. The micrometers read to 1 micron, and an experienced operator could duplicate readings to 2 or 3 microns.

The time required for filling the mold, setting the plugs, and focusing required about 15 min. The original length measurement was taken as soon as possible but the time varied from 15 to 20 min after the mixing was completed. The length changed rapidly during the first 4 to 6 hr and the readings were made frequently. Thereafter, the changes were small, and intervals between readings were extended.

The temperature of the room during the investigation varied from 70° to 80° F and the relative humidity from 20 to 80 percent. At the end of 24 hr the mold was removed and the specimen kept moist by burlap dipping in water at either side. Curing in this manner was continued for 28 days, except that it was necessary to remove the burlap during periods of measurement.

At the end of the damp-curing period the specimens were carefully measured and transferred to dry storage in the same room. Readings were made at intervals over a period of 3 months during dry storage, and then the specimens were immersed nearly to the top face in water and measured periodically.

## 4.3. Results of Tests

Length changes after the first 5 days in each storage condition were so small that the curves in figure 7 were drawn only for the initial periods. The maximum observed change for each test is stated on the graphs.

The basis for computing percentage changes for damp curing was the original length between reference points set in the plastic mortars about 15 min after mixing. The length at the end of damp curing was used as the starting point for dry-storage measurements, and the length at the end of dry storage was used as the starting point for moisture-expansion measurement (table 7).

It will be seen that the portland-cement mixtures (Nos. 1, 2, 3, and 4) did most of their shrinking in the first 6 hrs. Additions of lime or ground limestone resulted in extending the period over which rapid shrinkage occurred, as well as resulting in higher total shrinkage. A mortar in which
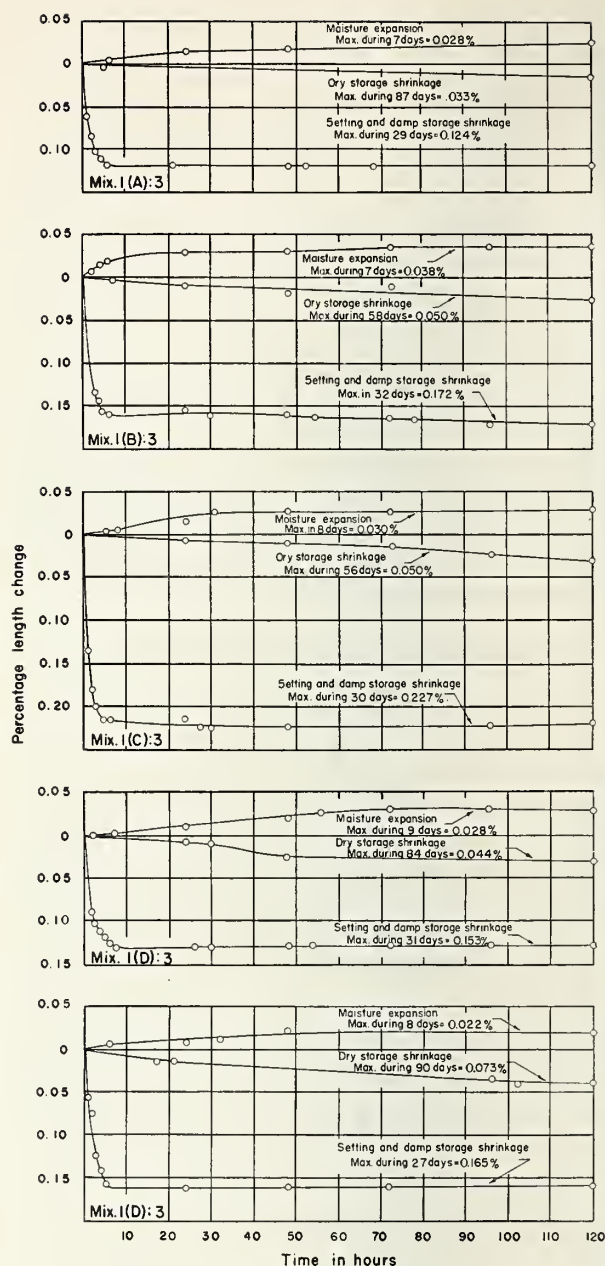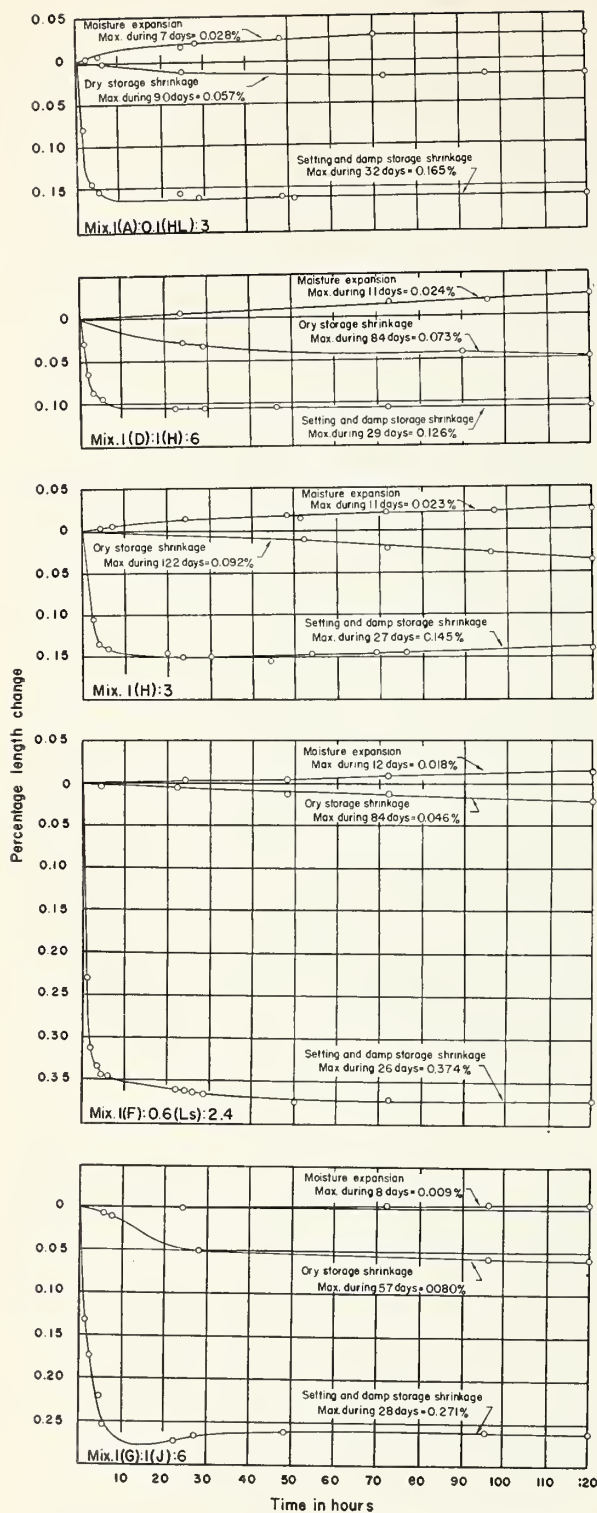
Left column graphs (Percentage length change vs Time in hours):

**Mix. I(A):0.1(HL):3**
- Moisture expansion Max. during 7 days = 0.028%
- Dry storage shrinkage Max during 9.0 days = 0.057%
- Setting and damp storage shrinkage Max. during 32 days = 0.165%

**Mix. I(D):1(H):6**
- Moisture expansion Max. during 11 days = 0.024%
- Dry storage shrinkage Max. during 84 days = 0.073%
- Setting and damp storage shrinkage Max. during 29 days = 0.126%

**Mix. I(H):3**
- Moisture expansion Max. during 11 days = 0.023%
- Dry storage shrinkage Max during 122 days = 0.092%
- Setting and damp storage shrinkage Max. during 27 days = 0.145%

**Mix. I(F):0.6(Ls):2.4**
- Moisture expansion Max. during 12 days = 0.018%
- Dry storage shrinkage Max. during 84 days = 0.046%
- Setting and damp storage shrinkage Max. during 26 days = 0.374%

**Mix. I(G):1(J):6**
- Moisture expansion Max. during 8 days = 0.009%
- Dry storage shrinkage Max. during 57 days = 0.080%
- Setting and damp storage shrinkage Max. during 28 days = 0.271%

Right column graphs:

**Mix. I(A):3**
- Moisture expansion Max. during 7 days = 0.028%
- Dry storage shrinkage Max. during 87 days = .033%
- Setting and damp storage shrinkage Max. during 29 days = 0.124%

**Mix. I(B):3**
- Moisture expansion Max. during 7 days = 0.038%
- Dry storage shrinkage Max during 58 days = 0.050%
- Setting and damp storage shrinkage Max. in 32 days = 0.172%

**Mix. I(C):3**
- Moisture expansion Max. in 8 days = 0.030%
- Dry storage shrinkage Max during 56 days = 0.050%
- Setting and damp storage shrinkage Max. during 30 days = 0.227%

**Mix. I(D):3**
- Moisture expansion Max. during 9 days = 0.028%
- Dry storage shrinkage Max. during 84 days = 0.044%
- Setting and damp storage shrinkage Max. during 31 days = 0.153%

**Mix. I(D):3**
- Moisture expansion Max. during 8 days = 0.022%
- Dry storage shrinkage Max. during 90 days = 0.073%
- Setting and damp storage shrinkage Max. during 27 days = 0.165%

Time in hours: 10 20 30 40 50 60 70 80 90 100 110 120

FIGURE 7. *Curves showing percentage-length changes of mortars during first 5 days of damp curing, dry storage, and total immersion in water.*

The maximum changes that occurred during the periods of test under the three conditions are shown in parentheses. The basis for computing percentage changes for damp curing was the length between reference points set in the plastic mortar about 15 min after mixing. The starting point for dry-storage shrinkage was the length at the end of damp curing and that for moisture expansion was the length at the end of dry storage.

the cementing material was a half-and-half mixture of portland and masonry cement (No. 23) gave a lower total shrinkage than three of the straight portland-cement mixtures, whereas a mixture of portland and natural cements in similar proportions (No. 27) showed a relatively high total shrinkage. Mortar 3, in which an air-entraining cement was used, gave the highest shrinkage of any of the straight portland-cement mixtures. Two separate batches of mortar 4 showed considerable difference in total shrinkage. Apparently some slight difference in the propor-

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TABLE 7. *Maximum shrinkage for two storage conditions and 7-day expansion in water*

| Mortar No. | Mortar mixture | Shrinkage— | | Total shrinkage after 4 months | Expansion after 7 days in water |
| | | After 28 days of damp curing | After 90 days of dry storage | | |
| | | % | % | % | % |
| 1 | 1(A):3 | 0.124 | 0.033 | 0.157 | 0.026 |
| 2 | 1(B):3 | .172 | .050 | .222 | .038 |
| 3 | 1(C):3 | .227 | .050 | .277 | .030 |
| 4 | 1(D):3 | .153 | .044 | .197 | .028 |
| 4 | 1(D):3 | .165 | .073 | .238 | .022 |
| 10 | 1(A):0.1(HL):3 | .165 | .057 | .222 | .028 |
| 23 | 1(D):1(H):6 | .126 | .073 | .199 | .024 |
| 7 | 1(H):3 | .145 | .092 | .237 | .023 |
| 20 | 1(F):0.6(Ls):2.4 | .374 | .046 | .420 | .018 |
| 27 | 1(G):1(J):6 | .271 | .080 | .351 | .009 |
| 21 | 1(F):1.2(Ls):2.4 | .402 | .064 | .466 | .029 |
| 22 | 1(F):2(Ls):2 | .487 | .048 | .535 | .037 |

TABLE 8. *Shrinkage of mortars by volume method*

| Mortar No. | Mortar mixture | Age | Total volume of shrinkage | One-third of volume shrinkage | Linear test values at 4 months |
| | | Weeks | Percent | Percent | Percent |
| 1 | 1(A):3 | 5 | 1.08 | 0.36 | 0.157 |
| 4 | 1(D):3 | 10 | .84 | .28 | .197 |
| 4 | 1(D):3 | 9 | .78 | .26 | .238 |
| 10 | 1(A):0.1(HL):3 | 12 | 1.35 | .45 | .222 |
| 13 | 1(F):0.1(HL):3 | 15 | .81 | .27 | ----- |
| 23 | 1(D):1(H):6 | 7 | 1.23 | .41 | .199 |
| 18 | 1(F):1(HL):3 | 17 | 1.35 | .45 | ----- |
| 22 | 1(F):2(Ls):2 | 14 | 1.44 | .48 | .535 |
| 1N | Neat cement | 4 | 4.20 | 1.40 | ----- |

tions, manipulation, or surrounding conditions during mixing and molding caused the different performance because it became apparent during the first few hours of the test.

## 5. Volume Shrinkage

### 5.1. Mortars and Specimens

Table 8 presents data on the total volume change of various mortars with respect to their initial volumes at the time they were molded. Most of the mortar specimens were cast from the same batches used in making bars for linear-shrinkage measurements.

The mold was made from a seamless brass tube 2.5-in. inside and 3.0-in. outside diameter. A length of 2.75 in. was used, and this was turned in a lathe to an inside diameter of 2.5 in. at one end and 2.75 in. at the other. The outside was then turned to give a uniform thickness of the sides of 0.04 in. The mold was made in this shape to facilitate the removal of specimens. The inner surface and ends were carefully ground with fine abrasives. The volume was obtained by setting the mold, small end down, upon a ground-glass plate and filling it with distilled water. A second ground-glass plate was used on top to determine the amount of water needed for complete filling. The weight of water divided by its density gave the volume of the mold, which was later used as the basis for computing volume changes of the specimens.

In molding specimens, one thickness of dry blotting paper was laid on a glass base plate and covered with one thickness of wet, fine-textured filter paper. The mold was then set on top of the papers, large end down, and filled with mortar by tamping in three equal layers. The top was then carefully leveled off and covered with a glass plate. It was necessary to leave the specimens in the molds for about a week, during which time they were covered with glass jars and stored in the same room as the bars for linear measurements.

After a week or more of curing at $75° \pm 5°$ F, the specimen was removed from the mold. Some specimens were easy to remove, and others had to be forced out. In most cases, there were several bubble holes in the sides, which had to be carefully filled with neat cement and smoothed. After filling the bubble holes the specimen was cured for 2 days in a damp cabinet.

### 5.2. Test Procedure

The only apparatus required for determining volumes of specimens is a sensitive balance of 1-kg capacity and accessories for weighing specimens in water. The equipment and procedure for weighing specimens in water is the same as described in ASTM Standard Method C97–47 for determining bulk specific gravity. Having determined the weight of a soaked specimen in the usual way and then its weight suspended in water, the volume can be computed by the formula

$$\text{Volume} = \frac{W_1 - (W_2 - b)}{d},$$

in which

$W_1$ = wet weight in air of specimen after soaking 48 hr,
$W_2$ = weight of wet specimen and hanger suspended in water,
$b$ = weight of hanger in water,
$d$ = density of water at observed temperature.

The determined volume subtracted from the volume of the mold is the volume shrinkage at time of observation.

The first volume determination of each specimen was made as soon as the specimen was completed and soaked for 48 hr, but usually this time was about 10 days after the mortar was mixed. The specimen was stored in a desiccator between times of subsequent measurements.

### 5.3. Results of Tests

The results of tests by the volume method on nine mixtures are shown in table 8. Five of these were taken from the batches mixed for linear shrinkage. Because volume shrinkage is nominally three times that of linear measurements, the

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volume measurements given in table 8 have been divided by three to simplify comparisons.

The balance used to determine volume changes was sensitive to 0.01 g, and 10 successive volume measurements on 1 specimen during a period of 3 days showed a standard deviation of 0.017. These volume determinations increased about 0.05 percent from the first to the last observation, which was probably due to moisture expansion of the specimens. The reproducibility of results is believed to be about 1 part in 20,000.

It was thought that the volume test specimen, because of less constraint by friction on the mold, would be less apt to form shrinkage cracks than the long specimens used for linear measurements. The measurements in the linear test were probably affected considerably by the fact that shrinkage undoubtedly occurred during the first 15 or 20 min while the reference plugs were being set and the microscopes focused. This early shrinkage could not be determined by that method. By contrast, the volume method determines total contraction from the time the mixture is placed in the mold, including slump.

Although the volume method has certain advantages in that simple specimens and apparatus can be used, the necessary filling of surface voids caused by air bubbles is an exacting and time-consuming task. Furthermore, the results are vitiated to some extent by moisture expansion while the specimens are soaking in water before each determination. This could be overcome by weighing the specimens in mercury instead of water, and the accuracy of measurements would be increased considerably.

## 6. Discussion of Volume Changes

Differences in curing conditions for the mortar specimens tested for linear shrinkage and volume shrinkage prevent an absolute comparison of results obtained by the two methods. It is evident from the curves in figure 7 that the greatest shrinkage occurs during the initial hardening period and that drying shrinkage of the hardened specimens continues slowly for a considerable length of time. The maximum values obtained by the two methods afford approximate comparisons if the volume shrinkage is corrected for moisture expansion during the 2 days of soaking before the volumes were determined. Such comparisons on identical mixtures indicate that one-third of the volume shrinkages average about 40 percent higher than those obtained by direct linear methods. The rapid shrinkage during the first 2 hr indicated by the curves in figure 7 shows that an appreciable amount of shrinkage must have occurred before the first reading of the microscopes could be made. If a correction for this were made, some of the differences would be eliminated. Probably the greatest difference is due to a large amount of the shrinkage being in a vertical direc-

tion. Vertical shrinkage is not ordinarily measured by the linear method.

For like conditions of curing it is probable that the two methods would give comparable results if by some means the early shrinkage could be included in the linear measurements, and the volume determinations were corrected for moisture expansion caused by immersion in water.

In order to illustrate the effect of shrinkage on measurements between reference points when cracking occurs, two clay specimens were made. A kaolin of low wet strength and a ball clay of high wet strength were used. Each clay was mixed with sufficient water to secure a consistency judged to be the same as that of a stone-setting mortar. Two shallow pans were filled with the mixtures and the top surfaces leveled with the tops of the pans. Several reference pins were set in each pan with a specially made gage, so that they were in line both longitudinally and transversely. The clays were allowed to dry in the laboratory for a week, then placed in an oven at 110° C for 2 days.

In figure 8, the top surfaces of the clays are shown with the pins encircled in ink. The lines at one edge of the pans indicate the original position of the transverse rows of pins. The high-strength clay, shown in the larger pan, contracted without cracking and some of the pins moved slightly out of line. The low-strength clay cracked, and there was little movement of the pins; however, in the second transverse row from the right, the middle pin moved considerably out of line. It appears that shrinkage divided the clay into sections, which shrank as units.

The shrinkage of the high-strength clay longitudinally was 11 percent as obtained by over-all measurements or measurements between the pins. Similar measurements on the other specimen indicated only 0.59-percent shrinkage. However, when the width of cracks, measured between pins, was added to the amount the pins moved, the result indicated a shrinkage of 5.0 percent.

Vertical shrinkage in the larger specimen was measured by placing a straightedge on the top of the pan and measuring from it to the surface of the clay. The top surface was concave and lowest at the center because of constraint by the sides and ends of the pan. Several measurements along the centerline indicated an average vertical shrinkage of 22 percent. An estimate of vertical shrinkage in the low-strength clay could not be obtained because most of the sections between cracks warped upward at the edges.

It is not likely that the difference in shrinkage vertically and horizontally is the same for mortars and clays, but evidence was found that much of the shrinkage in mortar is vertical. In specimens made for volume shrinkage the mortar slumped noticeably below the top of the mold. Most of the specimens were difficult to remove from the mold, which indicated only slight horizontal shrinkage. This was further demonstrated

FIGURE 8. *Shrinkage of clay specimens (top surfaces).*

The smaller specimen was made of a Delaware kaolin, the other a Kentucky ball clay. The pans were filled flush with the top and the pins (encircled) were set in line with the lines at the edges of the pans. In drying, the kaolin cracked and the pins moved very little. The ball clay shrank as a unit and the pins moved consistently. The movement of the pins in the kaolin indicates a shrinkage of less than 1 percent, whereas for the ball clay a shrinkage of 11 percent is indicated.

by setting some of the specimens in water nearly to the top while still in the mold. There was no capillary rise of water between the specimens and mold. One specimen required a load of 400 lb to force it out of the mold, which indicated that it was bonded to the metal.

Linear changes in cement products often called "volume change," have been measured chiefly by two methods. One of these involves setting reference marks in specimens as soon as the mixture is placed in a form and using micrometer microscopes to read the changes. This method was used by Pearson [4], Scholer and Dawley [5], and by the present authors. For this method, the specimens are always molded in a horizontal position, and no indication of vertical shrinkage is obtained. The horizontal movement of the reference points is accurately obtained, but the results may be in error if cracks form between the points or if shrinkage occurs before the first reading is made. A more common method is to set metal plugs or glass plates in the ends of specimens when being molded and to start measurements after the cement has hardened sufficiently to permit removal of the specimen from the form. This method disregards the shrinkage that occurs during the first 24 hr or longer. According to Scholer and Dawley [5], there is no shrinkage during this period if watertight forms are used and evaporation is prevented. A variation of the last procedure has been used by Palmer and Parsons [3], who measured the length of the mold and used this as the original length from which to compute later shrinkage. They obtained high shrinkage values as compared to those commonly reported for 1:3 mortars made of portland cement, masonry cement, or lime. Rogers

and Blaine [6] employed a method in which 1- by 1- by 8-in. specimens were molded in 6-gang vertical molds with glass plates on top and bottom of each specimen. As soon as the top plates were in position the mold was set under a battery of dial gages, which gave the amount of shrinkage at any desired time. Their measurements on 1:3 mortars in which the cementing material was portland cement, portland cement and lime, natural cement, portland and natural cement, or straight lime gave a large range of values. In each group the maximum values were near 0.50 percent, with some minimum values below 0.20 percent. The large range might be caused by crack formations between the points of measurement in some of the specimens. The small cross section of the specimens and friction between the mortar and sides of the molds may have constrained shrinkage.

Carlson [2] has studied shrinkage with particular reference to the effect of internal cracking. He advances the theory that all cement mixtures would shrink as much as neat cement if they were not constrained by aggregate particles. The compressibility of quartz and most other types of rocks is very small; the cement paste surrounding each sand particle is, therefore, in tension due to shrinkage. The result is that fine cracks are formed in the paste between the fine aggregate grains. Much of this shrinkage is not indicated by ordinary methods of measurement. Carlson made a specimen with rubber aggregate instead of sand or gravel and found that it contracted about eight times as much as ordinary mixtures.

Some studies made at the National Bureau of Standards on crack formations in terrazzo have a bearing on Carlson's theory. Terrazzo is made

15
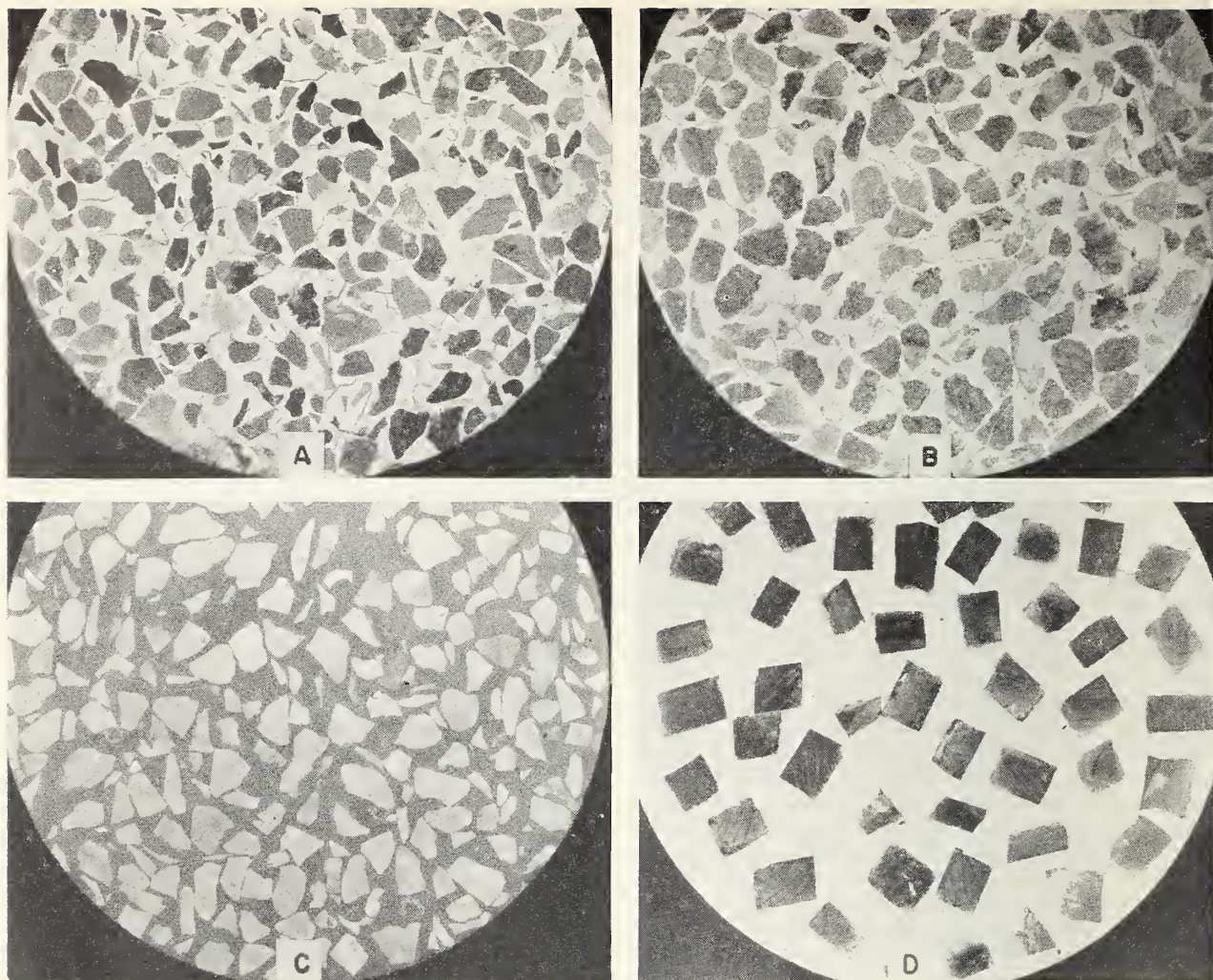
FIGURE 9.   *Terrazzo specimens.*

A and B were made of marble chips and a finely ground cement, whereas C was made of marble chips and a coarser cement. Specimen D was made of a finely ground cement and rubber aggregate. A and B cracked extensively, whereas C did not form visible cracks.   D cracked moderately near the outer edge
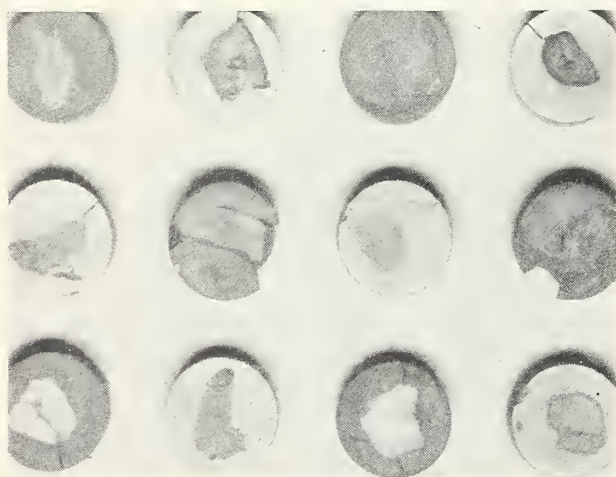


FIGURE 10.   *Shrinkage cracks in cement surround aggregate particles of low compressibility.*

Each specimen is a ½-in. cylinder of neat cement enclosing one fragment of marble.

of 1 part portland cement and 2 parts marble chips.   In figure 9, A and B show two specimens, made with a finely ground cement, which cracked extensively.   Specimen C was made with a coarser cement, and it has relatively few cracks.   Specimen D was made with a finely ground cement and a rather hard rubber aggregate.   No cracks are visible in the center area, but some developed in the outer part, possibly due to differential drying. All the specimens shown in figure 9 were damp-cured under identical conditions.

Each specimen shown in figure 10 was made with a single marble chip and portland cement molded into a ½-in. cylinder and damp-cured for 2 weeks.   All of these cracked badly.   This is not surprising because the linear shrinkage of neat cement, as determined by the volume method, is 1.4 percent.   In a mortar the sand particles are surrounded by neat cement, and it seems inevitable that fine cracks will form in great

numbers, although they may be too small to be visible.

# 7. Resistance to Freezing and Thawing

## 7.1. Specimens and Test Procedure

The mortar bars used for determining linear shrinkage were cut into prisms 2.5 by 2.5 by 2 in. after the shrinkage tests were completed. In order to adapt these blocks to accurate volume determinations, the bubble holes showing on the surface were filled with neat cement.

The dry weights, absorptions, and volumes of two specimens of each mortar were determined before the freezing tests were started. These were frozen at —15° C (free to drain in still air) and thawed in water at room temperature. Two cycles were made each day, and the specimens were in the freezing chamber at least 6 hr for each freezing. During the night or over holidays they were in the freezing chamber much longer. In the early stages of this test, the specimens were tested for volume changes after each 20 cycles, but when it was found that changes were taking place very slowly the periods between volume tests were extended. After 940 cycles had failed to produce any marked deterioration, a final examination was made and the test discontinued.

Sonic methods of determining changes in modulus of elasticity during the test could not be used because the specimens were too small.

## 7.2. Results of Tests

Volume changes, final strengths, and visual changes are reported in table 9. Absorption and weight changes were not of sufficient magnitude to justify conclusions, and these values are not tabulated. During the progress of these tests, it was found that the wet weights of specimens increased considerably, as is common in freezing tests on many other porous materials. At the end of the test these specimens were dried in an oven at 115° C for 10 days, and over half of them still weighed more than indicated by their original dry weights. One possible reason for this is that water forced into fine capillaries could not be driven off at ordinary drying temperatures. Further hydration of the cement may have accounted for some of the weight increase.

The original strengths of these mortar specimens were not determined, hence the values given in table 9 afford no basis for judging strength losses due to freezing. They are stated merely to show that the mortars did not suffer severe deterioration in the freezing test. Some of these can be compared with the results of compression tests made on specimens cast in metal molds and reported in table 1. These do not, however, afford reliable comparisons because specimens made in porous molds give higher test values, as reported in section 8.

## 7.3. Discussion of Results

All the mortars in this test showed excellent frost resistance, and the small differences indicated by the results are not conclusive. The high resistance to frost deterioration may have been due to the specimens being made in porous molds, which probably reduced the water-cement ratios of the specimens.

Judged by the decreases in volume and the final strengths, it may be assumed that the 1:3 mixture made with portland cement D and a similar mixture made with masonry cement H were the least resistant. Other 1:3 mixtures indicated satisfactory results. The mixtures containing ground limestone or hydrated lime gave indications of good frost resistance. This may be accounted for, in part, by the increased air entrainment caused by the addition of fine materials.

# 8. Effect of Different Curing Conditions

In some of the early tests of this investigation it was noted that specimens molded and cured in contact with porous materials indicated that such conditions were favorable to the mortars. This was interpreted to mean that the extraction of

TABLE 9. *Results of tests and observations on specimens after 940 cycles of freezing and thawing*

| Mortar No. | Mortar mixture | Decrease in volume | Compressive strength after freezing | Appearance | | Remarks |
| | | | | Cracks | Edges | |
| | | *ml* | *psi* | | | |
| 22 | 1(F):2(Ls):2 | 0.54 | 10,050 | None | Sharp | |
| 21 | 1(F):1.2(Ls):2.4 | .42 | 11,060 | do | do | |
| 20 | 1(F):0.6(Ls):2.4 | .38 | 11,230 | do | do | |
| 13 | 1(F):0.1(HL):3 | .31 | 8,140 | do | do | |
| 4 | 1(D):3 | .86 | 3,540 | do | do | |
| 4 | 1(D):3 | .77 | 3,200 | do | Rough | |
| 24 | 1(E):1(H):6 | .46 | 5,220 | do | do | Slight spalling. |
| 1 | 1(A):3 | .24 | 6,970 | do | do | |
| 27 | 1(G):1(J):6 | .16 | 4,970 | do | do | Slight spalling. |
| 2 | 1(B):3 | .24 | 8,380 | do | do | |
| 3 | 1(C):3 | .46 | 6,370 | do | do | |
| 7 | 1(H):3 | 1.05 | 3,190 | do | do | Slight crumbling. |

excess water from the mortars, while still plastic, was desirable and that mortars may perform in masonry better than might be expected when judged by results of laboratory tests. It was decided to experiment on specimens made and cured under conditions simulating those in masonry and also those in the usual laboratory tests for more definite comparisons.

Molds were made of limestone used for the sides and bottom to simulate the masonry condition and of wood to simulate the laboratory condition. Both types were designed for casting thin specimens on edge, so that each specimen would be surrounded by a 1-in. thickness of porous or impermeable material.

The limestone chosen for making the molds was the same as that used in the bond test specimens and described in section 2.3. The wood molds were greased inside to make them impermeable.

The mortar mixtures were of the same proportions as used in the bond tests, but some of the ingredients differed. New samples of cements E and F and sand from a different source were used. The sand all passed a No. 8 sieve, and the percentages retained on the Nos. 16, 30, and 50 sieves were 19, 38, 57, and 95, respectively. This sand contained more quartz grains, which were somewhat coarser and more angular than those of Potomac River sand.

The five mortar mixtures listed in table 10 were made and molded into specimens 4 by 5 by 1 in. The 4-in. dimension was vertical, and the mortars were tamped in 3 layers. Half of each mixture was placed in one type of mold and half in the other. The molds were left open for 2 hr and then covered with damp burlap, which was soaked once each day. Both types of specimens were thus cured in the molds for 1 month.

The specimens were first tested for flexural strength, then the two ends of each were cut into pieces 3 in. square for absorption and bulk density tests. Finally, 1-in. cores were cut and tested for compressive strength.

Results of all tests are shown in table 10, which also shows the number of tests that were averaged for each property. The results consistently show

that specimens made in porous molds were of superior quality. Such curing seems to be more favorable to the masonry cements than to portland cements.

# 9. Relation of Mortar to Performance of Masonry

Some of the undesirable performances of stone masonry are often attributed to the setting mortar. It is proposed to discuss some of these in relation to laboratory results and observations on structures. Evidently much can be learned from studies on existing structures that show that errors have been made in selecting or combining materials. Laboratory research cannot feasibly reproduce all of the conditions in masonry or the time factor. It is not to be inferred that the following statements are proved facts, but if they lead to further discussions and records of experiences from others, some good purpose may be served.

## 9.1. Spalling of Stone at Mortar Joints

Stone blocks sometimes spall at the mortar joints. The spalls are usually thin and feather out 2 or 3 in. away from the joints. The spalls usually occur at vertical joints in ashlar masonry. They appear to be shear failures due to temperature stresses. A common explanation suggested for such trouble is that the mortar was too strong.

Stonework erected in cold weather could be expected to develop temperature stresses in vertical joints in summer. A computation based on a temperature rise of 80 deg F indicates that stones of the usual thermal expansion and elasticity values could not be stressed more than 1,500 psi in compression, if the joints are well filled with mortar. Most building stones have compressive strengths, as determined by standard tests, of more than 5,000 psi.

In tests for compressive strength, the ends of specimens are placed in direct contact with the metal plates of the testing machine. Under such conditions, friction between the metal and stone

TABLE 10. *Comparison of curing conditions in wood molds and limestone molds*

| Mortar No. | Mortar mixture | Compressive strength [1] | | | Modulus of rupture [2] | | | Absorption [3] | | | Bulk density [3] | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Type of mold | | Ratio [4] | Type of mold | | Ratio [4] | Type of mold | | Ratio [4] | Type of mold | | Ratio [4] |
| | | Wood | Stone | | Wood | Stone | | Wood | Stone | | Wood | Stone | |
| | | *psi* | *psi* | | *psi* | *psi* | | *%* | *%* | | | | |
| 19 | 1(F):0.5(Ls):3 | 4,360 | 6,770 | 1.55 | 1,377 | 1,742 | 1.26 | 8.60 | 6.98 | 0.81 | 2.014 | 2.080 | 1.03 |
| 8 | 1(I):3 | 1,140 | 2,450 | 2.15 | 624 | 972 | 1.56 | 9.85 | 7.89 | .80 | 1.860 | 1.894 | 1.02 |
| 15 | 1(E):0.4(HL):3 | 4,760 | 6,850 | 1.44 | 1,473 | 1,840 | 1.25 | 9.30 | 8.01 | .86 | 1.986 | 2.044 | 1.03 |
| 7 | 1(H):3 | 580 | 1,040 | 1.80 | 303 | 521 | 1.72 | 11.25 | 6.55 | .58 | 1.745 | 1.789 | 1.02 |
| 5 | 1(E):3 | 2,960 | 6,140 | 2.07 | 1,065 | 1,638 | 1.54 | 7.62 | 5.61 | .74 | 2.021 | 2.049 | 1.01 |

[1] Each value is the average of either 6 or 7 individual test.
[2] Each value is the average of 3 tests.
[3] Each value is the average of 2 tests.

[4] Ratio of the test value of the specimens made in stone mold to that of the specimens made in wood mold.

is sufficient to prevent lateral expansion at the ends of specimens, and spalling is restrained. However, if some pliable material, such as lead or several thicknesses of blotting paper, is inserted between the specimen and the metal plates, both spalling and final failure will occur at lower loads than in the standard test.

A similar stress condition probably exists at the contact between stone and mortar in vertical joints because the thermal coefficient of expansion of mortar is greater than that of stone. When the temperature rises and causes compressive stresses at the joints, the mortar also expands perpendicular to the face of the masonry at a greater rate than that of the stone. Such a condition is probably more favorable to spalling than that in the compression test where pliable materials are used between the stone and metal.

Some engineers have emphasized the desirability of making mortars less rigid. Although the $E$ values of mortars can be varied considerably, it is necessary to make the mortar rather weak in order to obtain a value below 2,000,000. The mortar joints in ashlar masonry make up such a small portion of the wall that little is gained in this way, and weak mortars are not apt to have sufficient resistance to weathering. A more logical course is to make the mortar sufficiently strong for weather resistance and take proper measures to thoroughly fill the vertical joints.

It is a common practice to set blocks in ashlar masonry on a leveled bed of mortar and fill the vertical joints later. The edge of the vertical joint is stopped with a hemp rope, and a grout is poured in until the space appears to be filled. This method may not completely fill the joint, and in any case high shrinkage of the mortar will occur because of the large excess of water. Pointing usually takes care of leakage whether the joint is thoroughly filled or not, but it may bring about a condition favorable to spalling, especially if strong mortar is used. The pointing mortar should not be stronger than the setting mortar.

Occasionally, spalls at the edges of stone blocks can be accounted for by defects in the stone or by injury in the fabrication process. When spalls occur directly opposite each other on either side of the same joint, it is logical to assume that the trouble was due to overstressing of the stone.

## 9.2. Efflorescence and Discolorations

Some of the water-soluble salts causing efflorescence may come from the mortar, but in most cases more come from other materials in the wall. The salts exist in small amounts and have to be leached to the surface by water percolating through the masonry. The most feasible means of prevention is to stop the entrance of large amounts of water. Absorption from the face is not likely to be sufficient to cause appreciable efflorescence, unless there are open joints. Efflorescence on stone masonry is usually localized in areas where considerable water finds entrance because of faulty drainage or other causes. The capillary rise of moisture from the ground is a common cause of efflorescence in buildings without grade waterproofing.

Discolorations, occurring mainly on new buildings, are usually leached out of the walls in the same way as efflorescence salts. Frequently, the two troubles go hand in hand, and apparently the salts causing efflorescence aid in bringing out the discolorations. Probably there are many sources of the discoloring matter, but the most common is believed to be the stone. Brown stains occurring on sedimentary rocks are usually caused by organic matter, such as can be carried into stone ledges near the surface of the ground from decayed vegetation. The position of the stains on masonry is often near the mortar joints, and for this reason it is commonly believed that the staining matter is from the mortar. The alkalinity imparted to the percolating water by the cement in mortars is involved in the transfer of the organic stains from the interior of the masonry to the surface [7,8]. Cements of low iron content called "nonstaining cements" have appeared on the market. Since the staining matter is not iron, as formerly believed, the value of such cements depends largely upon their low alkali content. Some specifications for stone-setting mortars place a limit on alkalinity of the cements. The use of "nonstaining cements" for the setting mortar will not prevent discolorations if a large amount of normal cement has been used in the backing wall unless an effective water barrier between the facing and backing is provided.

A considerable amount of water passing through the masonry is necessary to bring out conspicuous discolorations, and proper precautions taken during the construction of walls will usually prevent such troubles. The tops of unfinished walls should be covered during rains to prevent saturation of the masonry. A simple expedient that is believed to be helpful is to provide frequent weep holes in the facing. These should be in the vertical joints so they can be sloped upward from front to back.

## 9.3. Calcareous Vein Formations

In damp locations where considerable water finds egress through small cracks, there are apt to be vein formations such as illustrated in figures 11 and 12. Because these veins are mainly calcareous, it seems likely that they result from calcium hydroxide dissolved from the cement and deposited where the water evaporates. When this hydrate is exposed to the air, it takes up carbon dioxide and changes to calcium carbonate. Frequently, such formations fill the original crack and continue to grow into adjacent masonry materials. Figures 11, A, and 11, B, show many cracks opposite each other in blocks of marble on either side of vertical mortar joints. Apparently, the cracks in which the veins originally started
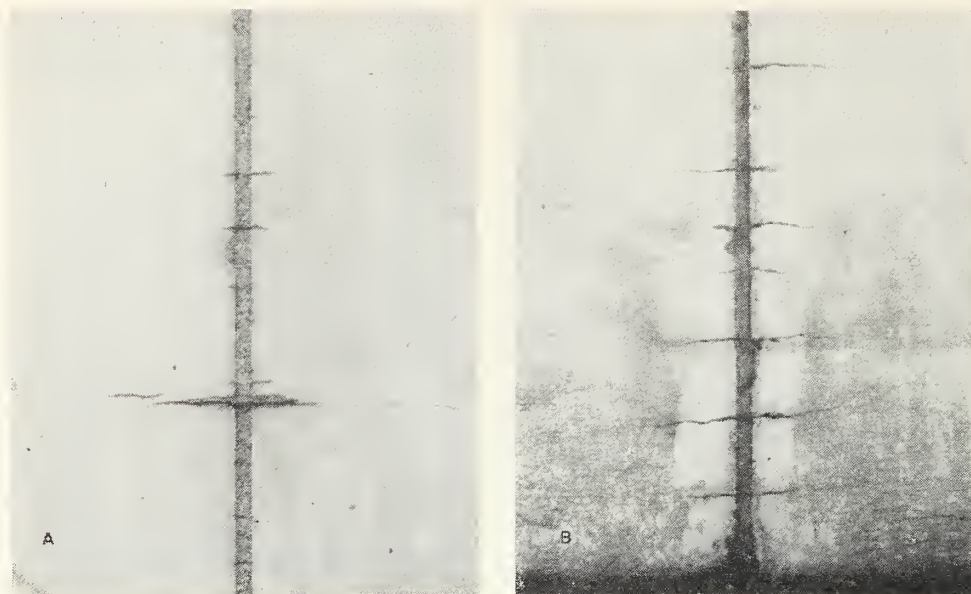
FIGURE 11.  *Calcareous vein formations.*

A, Calcareous vein formations
marble masonry that probably
started from shrinkage cracks in
the mortar joints; B, several cal-
careous veins starting from one
vertical joint between marble
blocks.



FIGURE 12.   *Calcareous vein formations in brick masonry similar to those in figure 11.*

The extensive migration of the formations is probably due to a large amount of free lime in the mortar joints
and considerable water seepage.

were cracks in the mortar. It has been demonstrated in the laboratory on a small scale that these growths can form in mortar cracks and continue into marble slabs, surrounded by the mortar, to start cracks.

Such vein formations are also found in other kinds of masonry. Figure 12 shows a brick wall with extensive cracks filled with calcareous growths. They are of common occurrence in concrete pavements, steps, and various other structures. There is a sufficient amount of lime in the cementing materials to form abundant growths if a considerable amount of water passes through the masonry. The solubility of hydrated lime is only about 1 part in 600 parts of water, hence the growths progress slowly but may become apparent within a few years.

## 9.4. Shrinkage Cracks

High shrinkage may cause cracks either between the stone and mortar, in the mortar, or across the joints. A mortar having a shrinkage of 0.2 percent should, theoretically, separate from the stone in vertical joints.[4] However, the results obtained in tests for bonding strength on a mortar of 0.5-percent shrinkage for specimens simulating the conditions in vertical joints were satisfactory, and no separations from the stone occurred. The reason is believed to be that the mortar was sufficiently slow-setting and plastic to enable it to adjust itself to the major shrinkage that occurs before the mortar sets. In the part of the experimental wall where a 1:3 lime mortar was used, it was necessary to wedge up each stone to prevent settling. The shrinkage of this mortar was not determined, but it was probably at least 0.5 percent. Under such conditions, it might be expected that the mortar would shrink away from the stone in both horizontal and vertical joints. So far as could be determined by observations this did not occur to any appreciable extent. Possibly the adhesion of the mortar in the plastic state was sufficient to maintain the bond and cause voids within the mass. Such effects have been frequently noted in plastic calking compounds where contact with the sides of joints was maintained but cavities formed within the plastic itself. Such compounds often shrink 20 percent or more by volume.

[4] The relation between stress, strain, and modulus of elasticity of an elastically deformed body is, $S = E\epsilon$, where $S$ is the force per unit of area, $E$ the modulus of elasticity, and $\epsilon$ is the unit deformation. $E$ for most mortars is at least 2,000,000 psi. Assume that the shrinkage of an unrestrained mortar is 0.2 percent. In a vertical joint, if the adjoining stones do not move, and if the mortar adheres to the stone and remains intact, its unit deformation $\epsilon$, upon drying, must be 0.002. Substitution of these values of $E$ and $\epsilon$ in the above equation results in a value for stress of 4,000 psi. Neither the tensile strength nor the bonding strength of the mortar to the stone is sufficient to withstand such a stress. It is apparent, therefore, that where such a joint remained intact, some deformation must have occurred while the mortar was plastic.

## 9.5. Surface Scaling

A phase of weathering that causes rapid surface deterioration of stone and other masonry materials results from crystallization of waterborne salts within the pores. The common source of the salts have been discussed in section 9.2. Aside from water-soluble salts originally in the masonry material, it is recognized that chemical reactions sometimes result in the formation of other salts that give trouble.

Efflorescence is commonly referred to as a formation of the salts on the surface where water solutions evaporate. Some of the salt crystals may form in the pores of masonry near the surface. The crystal growth in the pores produces stresses on the walls of pores sufficient to force off flakes of the stone. This type of decay can usually be readily distinguished from others by its appearance. Usually the masonry materials are not appreciably injured except near the surface, but if the conditions bringing about the action persist, the scaling may continue to flake off one layer after another until a considerable depth is reached. For this, a large amount of water has to enter the masonry, and there must be a considerable supply of the salts.

When there is a continual soaking of some part of the masonry, it is sometimes noted that scaling is not uniformly distributed over the dampened area. Different materials or even different parts of one material may disintegrate and others show little or no effects. A case has been cited by Schaffer [9] where limestone and sandstone were used in alternating courses and scaling was confined mainly to the sandstone. For such selective decay he mentions the possibility that the sandstone may have a higher capillary "pull" than the limestone, which enabled it to draw most of the salts away from the mortar joints. The same authority illustrates selective decay by what he calls soft beds in some kinds of stone, which cause the stone to scale more rapidly in certain zones than others. Such performance has been noted for some stones in this country, but such stones, in the original condition, seem to be of quite uniform hardness. In the exposure test wall some of the very hard stones have undergone pronounced scaling within a period of 4 years, whereas adjoining stones of various textures and densities have remained in perfect condition. Such results indicate that the capillary forces in some materials are strong and enable the materials to draw water solutions from adjacent materials.

Ordinarily one would not expect that a sufficient amount of salts would be present in thin mortar joints to cause scaling of adjoining stones. One sandstone building can be cited where bands of scaling have formed in the stone bordering the joints. This stone is very fine-grained and is known to have rather high capillary suction. In

this case the indications are that the salts were drawn from the mortar.

The rates of drying after the materials have been soaked may also have some significance in relation to the movements of salt solutions in a masonry wall. Twelve mortar mixtures were subjected to this test, and the results are given in table 11. The specimens were soaked by complete immersion in water for 24 hr, weighed, and exposed to air on all faces. The air temperatures varied between 75° and 85° F, and the relative humidity varied between 45 and 60 percent. The results are believed to be comparative for the mortars involved because they were all exposed at the same time and same location.

Evidently these mortars all dried slowly in comparison to the drying rates for most building stones. Most of the specimens retained 70 percent or more of the water for 24 hr. Sixteen limestones tested in the same way indicated much higher drying rates. The average loss of total water content was 25 percent in 6 hr and 75 percent in 24 hr. After 24 hr none of the specimens retained more than 45 percent of the water.

In this investigation a few studies were made to determine differences in pore structure in certain stones and a few mortars. One test (test A in table 11) consisted in measuring the rate of absorption during 1 min when only one face of the specimen was in contact with water, then after drying, measuring the 1-min absorption when all but one face was in water (test B). Under such conditions, one might expect that the absorption in test B would be greater in proportion to the areas in contact with water. For the oolitic limestones used in this test the absorption ratios were near the area ratios, but with the mortars this was not the case. When the area ratios for mortar specimens were more than 4:1 the absorption ratios were frequently less than 2.5:1. This seems to indicate that when a specimen is absorbing only from the bottom, air displaced by the water can

escape fairly readily from the sides, but when the bottom and sides are submerged there is less opportunity for the air to escape and absorption is delayed. The test is simple and is believed to give information on pore structure. Although there is an appreciable difference between the various mortars, as shown by the ratios, the results indicate that mortars generally have fewer connecting pores than the more commonly used limestones have.

## 10. Summary and Conclusions

1. Several mortar mixtures in which the cementing material was portland cement, masonry cement, natural cement, or combinations or portland with natural or masonry cement were studied. Lime or ground limestone was combined with several portland-cement mixtures.

2. Bonding strengths of 33 mortars to natural stone were studied. The bonding strengths to fine-grained stones were considerably higher than to coarse-grained stones. Sixteen mortars were tested for bonding strength in limestone joints simulating the conditions in both horizontal and vertical masonry joints. Eleven of these gave practically the same strength in both types of joints, and the other five averaged 60 and 40 psi, respectively, for the horizontal and vertical joints.

In 1:3 mortars of masonry cements or 1:3 mortars in which the cementing material was a half-and-half mixture of masonry and portland cements, the bonding strengths were rather low. The results indicate that 1:3 mortars of portland cement are apt to give variable bonding values in vertical joints. The addition of 40- or 50-percent hydrated lime by volume of portland cement or like amounts of ground limestone, to straight-cement mortars seems to improve the uniformity of bonding strengths in horizontal and vertical joints.

TABLE 11. *Absorption rates and drying rates of mortars*

| Mortar No. | Mortar mixture | Absorption rates [1] | | | Drying rates of saturated specimens | |
| | | | | | Water retained— | |
| | | Test A [2] | Test B [3] | B/A | after 6 hr | after 24 hr |
| | | $g$ | $g$ | | % | % |
| 22 | 1(F):2(Ls):2 | 0.68 | 1.44 | 2.1 | 87 | 80 |
| 21 | 1(F):1.2(Ls):2.4 | .31 | .71 | 2.3 | 91 | 84 |
| 19 | 1(F):0.5(Ls):2 | .32 | .78 | 2.4 | 92 | 86 |
| 13 | 1(F):0.1(HL):3 | .22 | .56 | 2.4 | 92 | 89 |
| 4 | 1(D):3 | 1.08 | 2.29 | 2.1 | 85 | 42 |
| 4 | 1(D):3 | 1.00 | 2.39 | 2.4 | 88 | 43 |
| 24 | 1(E):1(H):6 | .19 | .48 | 2.5 | 82 | 75 |
| 1 | 1(A):3 | .14 | .52 | 3.7 | 89 | 83 |
| 27 | 1(G):1(J):3 | .19 | .48 | 2.5 | 86 | 76 |
| 2 | 1(B):3 | .22 | .46 | 2.1 | 85 | 80 |
| 3 | 1(C):3 | .22 | .50 | 2.3 | 89 | 84 |
| 7 | 1(H):3 | .42 | 1.23 | 2.9 | 78 | 52 |

[1] One minute absorption in grams.
[2] Area in contact with water in test A = 6.25 in².
[3] Area in contact with water in test B = 26.25 in².

3. A study of methods for determining shrinkage of cement mixtures indicates that many of the published values does not include the shrinkage that occurs while the mortars are plastic. Mortars, when placed in contact with porous materials, shrink rapidly during the first few hours, and most test methods do not measure this early shrinkage. Linear methods do not indicate localized contractions that cause cracks between points of reference, nor do they indicate vertical shrinkage. Cracks between points of measurement may divide the specimen into sections that shrink as units. The cement paste surrounding rigid aggregate particles, such as the sand in mortars, inevitably cracks because the shrinkage of the paste is much greater than the compressibility of the aggregate.

Shrinkage measurements were made by two methods with the object of finding the total shrinkage of mortars from the time they were placed in the molds. Linear values determined by a comparator method ranged from 0.157 to 0.535 percent, whereas a method for determining volume shrinkage usually gave considerably higher values. Experiments with clay mixtures indicated that crack formations in specimens cause results of measurements to be too low and that shrinkage in the vertical direction, while the mixture is still plastic, is much higher than in a horizontal direction.

The addition of 0.4 or 0.5 percent of hydrated lime or ground limestone to 1:3 portland-cement mortars greatly increases the shrinkage, but the improved working qualities imparted by such additions enables the mixtures to adjust themselves to high initial shrinkage and give good bonding strength in vertical masonry joints.

4. Tests on mortar specimens molded in contact with porous materials indicated higher strengths and lower absorptions than those obtained on specimens made and cured by standard laboratory methods.

5. Freezing and thawing tests on 12 mortars, for which the specimens had been molded and cured in contact with porous materials, indicated satisfactory resistance values for all of the mixtures.

6. Some of the more common troubles in stone masonry, frequently attributed to unwise selection of mortars, have been discussed. A few of the usual weathering processes in masonry structures have been described and calcareous vein formations illustrated.

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